THE UNIVERSITY OF ALBERTA

DURABILITY TESTS ON AN ASPHALT STABILIZED SAND

A THESIS

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ABSTRACT

Some soils, in their natural states, are unsuitable for use in a highway base. The addition of an additive will often improve a soil's properties sufficiently that its use becomes feasible.

Durability may be defined as the ability of a material to withstand change or wear.

In this investigation a medium to fine grained sand was mixed with various amounts of a liquid asphalt and subjected to various conditions of moisture and temperature. A sand-portland cement mixture was subjected to some of the same conditions for comparison purposes.

The unconfined compressive strength of 2 inch by 2 inch diameter specimens was the primary basis utilized for comparing the different mixtures. Length changes in 2 inch specimens during alternate freezing and thawing or heating and cooling were also measured.

The effect of partial saturation and freezing and thawing on larger sand-asphalt specimens, as determined by the triaxial compression test, was also reported.

In the investigation, sand-asphalt specimens, cured at 100 F, were found to decrease in compressive strength with increasing asphalt content.

The compressive strength of sand with 8 per cent portland cement was found to be at least 3 times that of the highest sand-asphalt strength.

Freezing and thawing was found to have little effect on unconfined and triaxial compression test results, but partial saturation had a large effect on the strength of sand-asphalt specimens.

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CHAPTER I INTRODUCTION

The Problem

With the increasing demand for high quality roads throughout Alberta convenient sources of high quality aggregates are being exhausted. However there are many areas where a lower quality aggregate is readily available. The problem is to determine an economical means of stabilizing the local materials so that their strength is adequate for the increasingly heavy highway loads. To be economical over a period of time, the highway obviously must be durable. *

Stabilization may be broadly defined as a process, mechanical or otherwise, which will improve the qualities of the soil. Many materials have been used as additives to improve natural soils. These stabilizing additives may be grouped into three classifications depending on their stabilizing mechanism.

The first group of materials cement the soil particles, and includes cements, limes, and acidic phosphorous compounds.

The second group are soil modifiers or conditioners, and includes lime, calcium, or sodium chlorides, and various surface active materials, which react with the soil changing its structure and texture and thus changing its engineering properties.

The third group are asphalts, coal tars, and some

*Durability may be defined as the ability to resist change or wear. In a highway base this implies resistance to weathering and traffic wear.

resinous materials which act as waterproofing agents, and on coarse grained soils may also cement the particles together.

Due to the extremely large variety in characteristics and properties of soils no general rules can be laid down specifying the percentage of an additive necessary to satisfactorily stabilize a soil. This means that every stabilization project must be individually investigated and / or local experience must be relied on in design.

There are several methods in use for design of stabilized bases. Many of these are based on empirical methods and local experience which has shown that the resulting product is satisfactory. Within the past decade the triaxial compression test has gained increasing favor as a method of analysis of stabilized materials. Empirical methods of thickness design using information from triaxial compression tests are presently used by some agencies in the U.S.A.

The design of an additive stabilized base usually includes some consideration of durability. In some cases this consideration may only extend so far as to assume that the selected mix will serve satisfactorily, and in other cases, some effort may be made to simulate in the laboratory the conditions of saturation or freezing and thawing which may occur in the constructed base.

This investigation was part of a continuing investiga-

tion of the stabilization of soil with additives. Pennell, in a companion study, used the triaxial compression test to observe the strength characteristics of a sand asphalt mix. The present investigation was an attempt to investigate the durability aspects of a sand stabilized with liquid asphalt.

Limitations of the Investigation

The large number of variables affecting the strength of a remolded soil also apply to an additive stabilized soil. In addition, a number of new variables are introduced, such as: type of additive, amount of additive, degree of mixing, effect on soil structure, and change in properties of the additive with time, to mention a few. For this reason, in all investigations some of the variables must be either made constant or ignored.

In this investigation the unconfined compression test was chosen for the strength testing because of its simplicity and its extensive previous use in this field. A simple test was necessitated by the large number of specimens involved and limitations on time. A limited comparison was made of unconfined test results with triaxial compression test results.

The testing program was conducted using one sand and one liquid asphalt. A constant method of mixing, compacting and curing was adopted for this as well. After a study of various tests adopted by the American Society

for Testing Materials, the American Association of State Highway Officials, and Watt in his work at the University of Alberta, procedures for durability tests were selected. These tentative procedures were modified in some cases as testing progressed.

There is a definite lack of information, both locally and generally, concerning conditions of moisture, temperature, and cycles of freezing and thawing in stabilized bases. No correlation with specific field conditions has been attempted. However, some tests were modified slightly to better approximate the local climatic conditions, and un-realistic conditions were discussed in an attempt to evaluate their significance.

Organization of the Thesis

The second chapter is a brief review and commentary on literature concerning asphalt stabilization. The emphasis is placed on durability, and particularly on new methods of testing.

Chapter three describes the materials employed in this project.

The fourth chapter describes and outlines the testing program carried out. A detailed procedure of specimen preparation and all tests is given, along with a discussion of each test. A description of the methods employed in analyzing data and results is also included.

Chapter five contains the results of the laboratory

testing. The results are presented in summary form and the values obtained, trends established, and the significance of each test's results are reviewed.

The final chapter contains conclusions and recommendations drawn from the testing program.

The appendix consists of data obtained in the testing program but of lesser importance, sample data sheets and a summarization of results.

CHAPTER II

Durability and Strength in Asphalt Stabilized Soil

A Literature Survey

Stabilization of soils with cutback asphalt is normally accomplished by blending the soil, at its optimum moisture content for compaction, with the cutback. The mixture is then compacted, and cured by allowing water and the solvent to evaporate. Since water and asphalt do not mix, the compacted mass is composed of a soil skeleton with the voids filled with asphalt globules which are surrounded by water. As the water and solvent evaporate, the asphalt globules ideally would cover all of the soil particles. (Michaels and Puzinauskas, 1956)*

During mixing, water facilitates the uniform distribution of asphalt throughout the soil. Asphalts are useful in stabilization because of their cementing and waterproofing qualities. The cementation property is considered to be the most effective in increasing stability in noncohesive soils such as sand, while the waterproofing quality is most useful in cohesive soils or soil-aggregate mixtures. (Katti et al 1959)

McLeod, (1943) states that a mechanically stabilized base, waterproofed with 1 to 2 percent of asphalt will have in situ, an equilibrium moisture content of 2 to 3 percent, depending on climate. This compares with an

* References are cited by indicating the author and the year of publication. The references are contained in the bibligraphy at the conclusion of this thesis.

equilibrium moisture content of about 7 per cent without waterproofing. Since the strength of the base is sensitive to water content, the value of the asphalt is in its waterproofing function.

The load-deformation characteristics of asphaltic mixtures are dependent on temperature. Asphalt is plastic at normal temperatures, but becomes more brittle as the temperature is decreased. Rader (1935) proposed that the causes of cracking in asphaltic pavements were the same as for concrete pavements. That is, contraction of the pavement caused by decreases in temperature and moisture content cause increases in tensile stresses until a failure occurs. Increased density due to better compaction greatly increased the modulus of rupture, but had little effect on the modulus of elasticity. Therefore, increasing the density of the material developed greater tensile strength at low temperatures.

In further work, Rader (1937), found that a high modulus of rupture indicated high tensile strength at low temperatures, and that a high modulus of elasticity indicated higher stresses in the mixture upon contraction.

Hubbard (1929) realized that a high degree of stability could be obtained by using a much lower percentage of asphalt than experience had shown to be advisable. However the resultant pavement could possess the undesirable

characteristics of a rigid structure.

Rice and Goetz (1949) investigated the effects of water immersion on the unconfined compressive strengths of various Indiana sands and liquid asphalt mixtures.

In preparing their 2 inch diameter by 2 inch height specimens all materials were heated to 110F, mixed in a Hobart mixer for 2 minutes, loose cured for 30 minutes and compacted under 3000 psi. static load. Specimens were cured for 5 days at 110F before being tested in compression. To determine the resistance to water, some specimens were immersed in water for 5 days at 110F prior to testing. Unconfined compression testing was done at a rate of strain of 0.05 inches per minute at a temperature of 110F.

Rice found that the amount of fines present in the sand was the characteristic showing the most consistent correlation with mixture strength when immersed.

In general, increasing the asphalt content resulted in greater density of the sand in the mix, less water absorption and less percent loss in strength. The water absorption varied as the voids in the mix, the percent voids filled with water being approximately constant for each sand.

For a criterion of suitability Rice used a minimum dry compressive strength of 80 psi, and allowed a maximum of 25% loss of strength after immersing. Results generally showed that the maximum wet strength occured at the same

asphalt content as the highest dry strength.

The results of the compression testing are not directly applicable to field design due to lack of field correlation but the test itself seems to be sensitive to small variations in mixture characteristics. Durability testing has commonly consisted of freeze-thaw or wet-dry tests.* The amount of material lost when the specimens are brushed is used as the durability criterion. The faults of the brushing test are the personal element involved, and the fact that the test is artificial, having little or no relation to any conditions occuring in a pavement structure. Therefore the severity of the test is often questioned since loose particles are always removed, completely exposing sound material to the freezing and thawing or wetting and drying in each cycle.

The soniscope showed promise as a means of non-destructive, dynamic testing for durability in research by Whitehurst and Yoder (1952). The velocity of pulsed vibrations sent through a material are measured by the soniscope. It appears that changes in pulse velocity are highly indicative of changes in the quality of specimens subjected to a cycling test such as wetting and drying or freezing and thawing. As a specimen deteriorates in a freeze-thaw test, the velocity of the wave through the specimen will decrease. Thus the pulse velocity after freezing and thawing or wetting and drying, relative to the initial pulse velocity through a specimen, can be used as a measure of the specimens durability.

* Such as ASTM D559-57 and ASTM D560-57.

A higher velocity through a specimen indicates a higher strength and modulus of elasticity.

Meyer (1952) used the soniscope for the testing of concrete pavements. The sound velocity was used as a measure of the modulus of elasticity which, while not an actual measure of strength, indicates the quality of the concrete. In the sound-velocity method, the velocity of a sound wave through the material is measured. If the density of the material and Poisson's ratio are known, the modulus of elasticity of the material can be determined from the equation:

$$E = \frac{V^2 (1+u)(1-2u)d}{(1-u)(32.2)\ 144}$$

where

E=modulus of elasticity - psi

V= velocity of sound-feet per second

d= density - pounds per cubic foot

u= Poisson's ratio

The velocity of sound varies from one pavement to another and also varies throughout a given pavement. Therefore it is important to work between the same points in testing the deterioration of a field test section.

The changes in sound velocity under various stress conditions were small enough to indicate that any stress conditions existing in a concrete pavement would not perceptibly effect the velocity of sound as determined with the soniscope.

Sound velocities measured by Meyer were all in the 12,000 to 16,000 feet per second range, which is supposedly the 'good concrete' range. However some of the pavements were obviously deteriorated and therefore the value of the sound-velocity method of determining pavement deterioration was not fully determined.

Another sonic method used for determining the modulus of elasticity is known as the resonant frequency method. This method was used by Goetz (1955) for determining the modulus of elasticity of bituminous mixtures. It was used only on small, properly proportioned laboratory samples, while the sound velocity method was not restricted in this manner. Goetz determined the modulus of deformation by the resonant frequency method to be about ten times that found by beam deflection methods on the same samples. However the author states that the strain rate is very much higher in the resonant frequency method so the results are not necessarily invalidated. In asphalt pavements it seemed that, from the standpoint of resistance to cracking at low temperatures, it was desirable that a mixture possess a low modulus of elasticity and a high strength.

Abbott and Craig (1960) have used the resonant frequency method (sonic) for evaluating stripping characteristics of cutback asphalt mixes. Curing was done at room temperature, with periodic determination of weight and fundamental frequency. The loss of solvent from the specimens

was very slow, however the sonic modulus (modulus of elasticity determined by sonic methods) increased more rapidly than the rate of solvent loss indicated. It was therefore concluded that changes in the asphalt occured during the curing period that hardened it to such an extent that it was near an asphalt cement mixed specimen in sonic modulus, even though part of the asphalt remained in solution. When specimens were immersed a very pronounced decrease in sonic modulus was found. It was found that the water resistance of cutback and asphalt cement mixes was about the same when specimens were at the same elasticity (i.e. the same sonic modulus.) However, the cutback mixtures take a long time to dry out and reach this modulus, and during this time were susceptible to severe stripping. There are additives available that protect against this stripping and can make the cutback mixtures less stripping susceptible than similarly treated asphalt cement mixtures.

A vibratory type machine developed by the Royal Dutch Shell Company was used by Maxwell (1960) for testing pavements and unsurfaced soils. With this machine, pavement evaluation was done in two distinct manners. In the first approach, the machine was used as a source of vibrations and the wave velocity was measured as in previous work (Meyer 1952). In the second approach a relationship is found between a dynamic load and the resulting deflection of the pavement surface under a circular loaded area.

Correlations can probably be developed between the values determined from vibratory measurements and conventional design such as unconfined compressive strength, subgrade modulus, density, and California Bearing Ratio. Periodic measurements would likely also provide information on seasonal changes in a pavement structure such as wetting and drying or freezing and thawing.

The ability of asphalt pavements to withstand repeated flexure and stress reversals could be a durability consideration. Monismith (1958) defines flexibility as a pavement's ability to bend repeatedly and to conform to differential movement in the subgrade or base. It was proposed that under the numerous repetitions of stress, a wearing surface could strain harden, decrease in ductility and eventually crack. The effect of load repetitions on beams composed of crushed granite and an 85 to 100 penetration asphalt cement were investigated. Fatigue resistance was found to increase with increasing asphalt content. For a given load magnitude, a dense graded mixture was found to have greater fatigue resistance than an open graded mixture.

In an investigation of thixotropic characteristics, Monismith and Secor (1960) found that changes in the asphalt occuring over a short period of time were not related to changes in the strength characteristics of the aggregate asphalt mixture when subjected to cyclic loading. In

further testing, Monismith, Secor and Blackmer (1961) found that the stress reversals showed little effect on fatigue behaviour if maximum bending strains were the same in both directions. Earlier findings on the effect of asphalt content and frequency of loading were also confirmed.

Sound velocity and resonant frequency methods are being extensively used in highway research. These methods have an advantage over many conventional tests in that they are non-destructive. Sonic methods have been used for observing the deterioration of laboratory freeze-thaw specimens and of highway pavements. Both sound velocity and resonant frequency methods can be used to determine the modulus of elasticity of concrete and bituminous materials. Sonic methods have also been successfully used to investigate the stripping characteristics of soil-cutback asphalt mixtures. Equipment developed in the Netherlands has been used in Europe and the United States for investigation of all elements in a pavement structure. Sonic and resonant frequency equipment does not measure engineering propert= ies directly but research to date indicates that a correlation with the physical properties of materials can be obtained.

CHAPTER III MATERIALS

Sand

The sand employed was a medium to fine grained sand according to the MIT grain size scale, obtained from the Department of Highway's McGinn Pit No.2.

The sand was sampled, split, and bagged by the Department of Highways and was used as required from the bags without further mixing or splitting. The moisture content of the bagged sand was less than 0.5 percent. Properties of the sand are described in Table I.

Asphalt

The asphalt used in this testing program was a medium curing liquid asphalt, designated MC 3, obtained from Husky Oil and Refining Ltd., Lloydminister, Saskatchewan. The supplier's data on the liquid asphalt is shown in Table II.

Cement

Type I portland cement, obtained from Inland Cement Company Limited, Edmonton, was employed in the soil-portland cement specimens. The mean seven day cube strength (ASTM C109) was approximately 3400 psi.*

Water

Distilled water was employed in sample preparation and ordinary tap water was used in the absorption tests and in soaking the samples prior to the freeze-thaw tests.

*Determined by the Highway's Division of the Research Council of Alberta.

TABLE I

SAND PROPERTIES

Pit location:	S.E. 5-53-1-W.5th
Specific Gravity:	2.66
A.A.S.H.O. Classification	A-3
Standard A.A.S.H.O. Dry Density	104 lbs. per. cu.ft.
Optimum Moisture Content	15%
Uniformity Coefficient	2.33

SIEVE ANALYSIS

U.S. Standard Sieve Series	Percent Passing
10	100.0
20	99.9
40	98.3
60	74.5
100	22.3
200	7.3

TABLE II

ASPHALT DATA

Specific Gravity at 60F	0.9792
API Gravity at 60F	13.0
Flash T.O.C.	150 + F
Water	Nil
Saybolt Furol Viscosity at 140F	392 seconds

Distillation:	% of Total over at:		% of Distillate Total over
	437F	0	0
	500F	2	9.3
	600F	14.5	69.1
	680F	21.0	100

% Residue to 680F. volume by difference 79.0

Residue: Pen. at 77F. (100 grams, 5 seconds) 155

Oliensis Spot Test (15% xylene) Negative

Soluble in Carbon Tetrachloride 99.8+%

Ductility at 77F (5 cm. per minute) 100+Cm.

CHAPTER IV TESTING PROGRAM

Preliminary Testing

Some investigation was necessary, before the testing program was undertaken, to eliminate some variables and to establish test procedures. To facilitate the comparison of results with the current triaxial investigation it was necessary to determine the compactive effort necessary to achieve standard A.A.S.H.O. density in the smaller size specimens employed. There was some doubt as to whether or not specimens should be cured before testing. Cured and un-cured specimens were subjected to a short freeze-thaw test and strengths were compared to control tests. The effect of curing time on strength was also investigated.

Compactive Effort

The standard A.A.S.H.O. maximum dry density of the sand was found by Pennell to be 104 pounds per cubic foot at a moisture content of 15 per cent. The compactive effort required to give this density at the same moisture content in 2.8 inch by 6.5. inch specimens and in 2 inch by 2 inch specimens was then determined. The required density was obtained with 2 inch specimens by using 36 blows of a 5.0 pound hammer dropping 12.0 inches. A comparison of the unit compactive energies in foot-pounds per cubic foot necessary to obtain the same densities in the different size molds is shown below.

AASHO 4 inch mold	2.8 inch diam.	2.0 inch diam.
12,400	34,200	49,500

These figures indicate that the compactive effort was utilized less efficiently in the smaller diameter molds.

Cured and Uncured Strengths

Twelve specimens were made at each of 1 per cent, 3 per cent, 5 per cent, 7 per cent, 9 per cent, and 11 per cent MC-3. Six specimens were oven cured at 100 F for 100 to 120 hours, and then soaked for 24 hours. Three of these were then tested in compression. The remaining three were sealed in polyethylene bags and subjected to 7 cycles of freezing and thawing.

The six remaining specimens were treated identically except that there was no curing period and no soaking.

Curing Time

A number of specimens were made at a MC-3 content of 5 per cent and water content of 12.5 per cent and tested after curing at 100 F for times varying from zero to 150 hours. From a graph of unconfined compressive strength versus curing time, the test curing time of 120 hours was selected.

Testing Program Outline

Durability tests on 2 inch diameter specimens mixed at optimum moisture contents, determined by Pennell, with 3, 5, and 7 per cent of MC-3 constituted the major portion of the testing program. The unconfined compressive strength was used as the criterion of durability.

Six durability tests were employed, as well as control

tests for one asphalt content. Six specimens were used per asphalt content for each test. The tests are identified as follows;

Test 1 Control

Test 2 Capillary Absorption Test

Test 3 Immersion Absorption Test

Test 4 Heating and Cooling Test

Test 5 Closed Freeze-Thaw Test

Test 6 Open Freeze-Thaw Test

Test 7 Closed Freeze-Thaw Test with no evaporation allowed during freezing and thawing.

Test 8 Control, with soaking.

The optimum mix design for the McGinn Pit No. 2 sand stabilized with portland cement was obtained from the Research Council of Alberta. The mix used was identical to that used for the stabilized base on Highway 16 near Stony Plain, Alberta. The unconfined compressive strength was used as the criterion of durability.

Two control tests, and four durability tests were conducted. Since individual durability tests were nearly identical to tests on the asphalt stabilized material similar numbering is employed. The tests used are identified as follows:

Test 1 Control - 7 day curing

Test 1A Control - 17 day curing

Test 3 Immersion Absorption Test

Test 4 Heating and Cooling Test

Test 5 Closed Freeze-Thaw Test

Test 6 Open Freeze-Thaw Test

Nine 2.8 inch diameter by 6.5 inch height specimens were prepared with each of 3, 5, and 7 per cent MC-3. These specimens were subjected to 10 cycles of freeze-thaw in a closed system with no evaporation allowed during freezing or thawing. The specimens were then tested in triaxial compression.

Specimen Preparation

The detailed procedure employed in preparing the unconfined compression and triaxial compression test specimens is given in Appendix F.

The procedure for sand-asphalt specimens consisted essentially of mixing sand with a pre-determined amount of water, adding MC-3, and compacting. The method of compacting 2 inch specimens was described earlier in this chapter. After compacting, specimens were cured for 5 days at 100 F. The larger triaxial test specimens were compacted in 5 layers, and cured for 7 days at 100 F. Sand-portland cement specimens, 2 inches in diameter by 2 inches high, were prepared similarily to the sand-asphalt specimens. They were then moist-cured for 7 days.

The term 'volatile' as used in this report applies to evaporable water and cut-back in the asphalt. The volatile content of each set of 6 specimens was determined from one sample taken from the loose mix after half of the specimens

had been compacted. The sample was considered dry after 24 hours in a forced draft oven at 110 C. If conditions had been ideal, the volatile content for all sets of specimens of one asphalt content should have been identical. This was not the case, as is evidenced by the values in Tables IX to XVI in Appendix B. The reasons for variations in volatile content are as follows:

1. The initial moisture content of the sand may not have been uniform. While several trials showed consistent values of 0.5 per cent moisture, there may have been slight differences in the numerous bagged samples from which the sand was obtained.

2. Volatile content may have varied within a mix. Even though all batches were mixed for about the same time and appeared to be well mixed there was doubtless some variation in moisture content throughout the mix. However, no data is available to support this hypothesis.

3. The percentage of cut-back in the MC-3 may have varied due to prolonged heating of the asphalt. There is no evidence to support this statement except that, on one occasion, the asphalt appeared more viscous than when previously used.

The total mixing time of about 7 minutes appeared to be adequate for all mixes. At higher moisture contents the MC-3 was more uniformly distributed throughout the sand. Hand mixing was used in this testing program, but, since no difficulty was experienced in mixing, a mechanical mixer would

work very well with this particular sand.

Calculation of the total aggregate voids filled with water and asphalt after curing at 100 F, compared with the total voids filled with residual asphalt, shows that the volatiles remaining in the specimens range from 2 per cent to 4 per cent. This low variation indicates that the curing time was adequate for a fair comparison of the various mixes.

No comparison with the required field curing time was possible due to the many possible differences between laboratory and field conditions. Laboratory conditions were:

1. 100 F plus or minus 1 degree with a forced draft and a low humidity for 120 hours.

2. Evaporation was possible from the entire surface area.

Under construction conditions, curing conditions may vary widely. The temperature and humidity are not uniform around the base and also vary throughout the day. The average curing temperature may be considerably less than the 100 F temperature employed in the laboratory. Evaporation is possible only from the top, and the surface area to volume ratio will be much less than under laboratory conditions. A comparison of the surface area exposed to air, to volume ratio (A/V), considering laboratory 2 inch by 2 inch specimens and a 2 inch layer in the field, shows that the laboratory A/V ratio is 3, compared to an A/V ratio of 1/2 in the field.

Control Tests 1 and 8

Six specimens were prepared at each asphalt content for comparison of unconfined compressive strengths with the strengths of durability specimens. After the 5 day curing period these specimens were weighed to 0.1 grams and the diameter and length were measured in the apparatus shown in Plate 1b. The specimens were then tested in unconfined compression as described later in this chapter.

Test 8 was identical to test 1 except that after curing the specimens were soaked in water at room temperature for 5 days before testing.

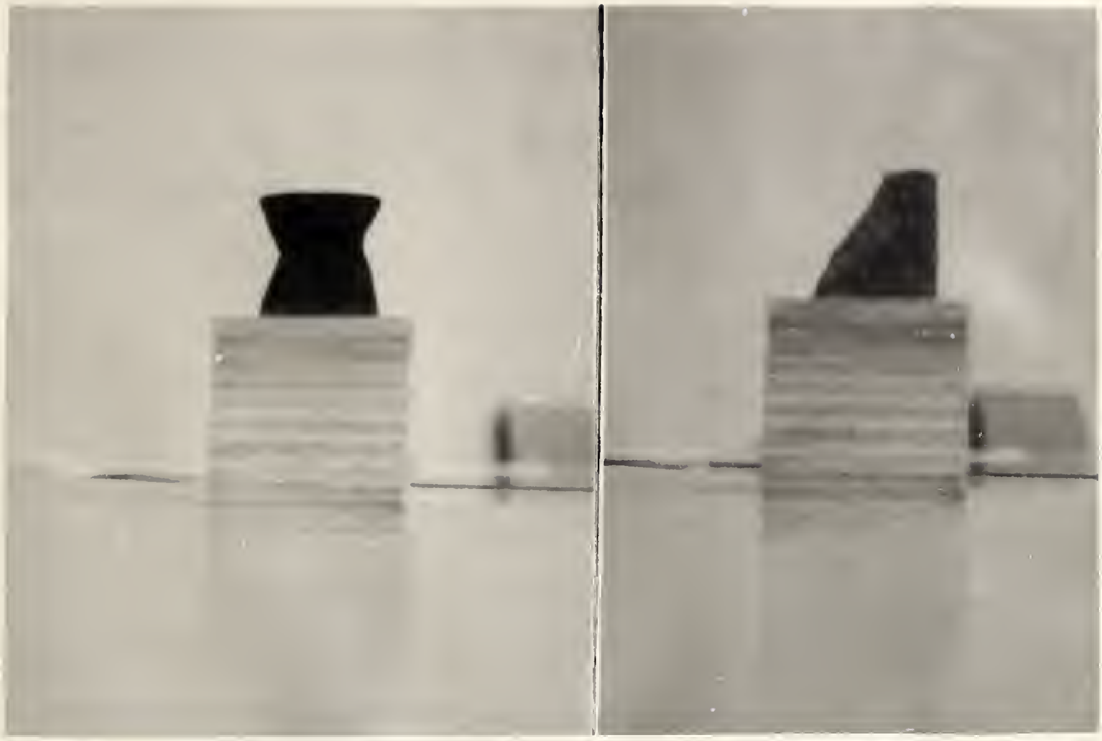
For measurement of length and diameter an extensometer was attached to a retort stand and fitted with a 5/8 inch diameter horizontal plate. Specimen length was taken as the average length, measured erect and inverted. The mold diameter was used for specimen diameter.

The first control test was intended to show the maximum compressive strength for each asphalt content. The other control test (Test 8), in which specimens were soaked 5 days, was included since freeze-thaw specimens were soaked for 5 days before testing. It was therefore necessary to determine what part of the resultant loss in strength was due to this soaking.

Durability Tests 2 to 7

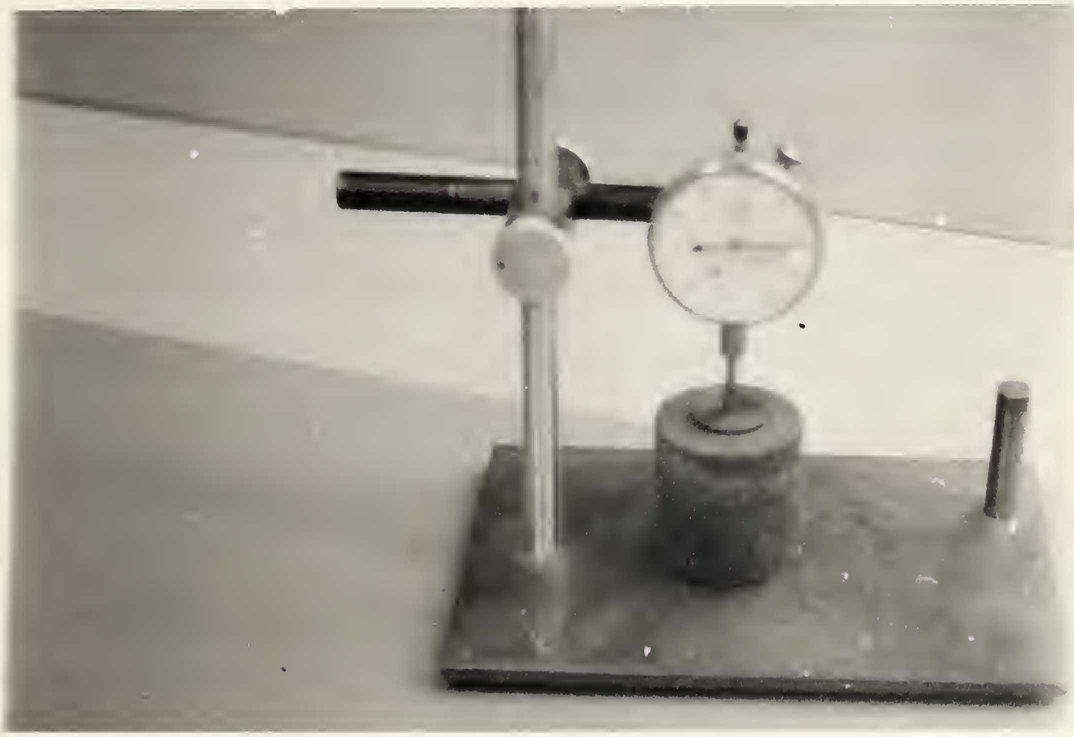
Test 2 Capillary Absorption Test

After 5 days of curing, 6 specimens of each asphalt content were placed on perforated lucite discs supported



Sand with 5% MC-3 Sand with 8% Cement

(a) TYPICAL FAILURES



(b) LENGTH MEASUREMENT

PLATE 1.

1 inch above the bottom of a large pan. The pan was then filled with tap water to a height of $\frac{1}{4}$ inch above the bottoms of the specimens. The test was conducted in a moist room and the specimens were covered with a polyethylene sheet to retard evaporation. Each day the water level was checked and the specimens were weighed. At the end of 14 days the specimens were measured and tested in unconfined compression.

The test procedure employed was similar to that given in ASTM(1958) page 455 except that specimens were 2 inches high instead of 1 inch, the moisture room was used instead of an absorption cabinet, and about $\frac{1}{4}$ inch of the specimen was immersed.

The value of this test is very doubtful for the material used since there was no raising of water in the specimens that could be attributed to capillarity. The only effect apparent was absorption of water by the portion of the specimens which was immersed.

The partial soaking of the specimens influenced the unconfined compressive strengths, since $\frac{1}{4}$ inch of the specimens had been soaked for 14 days.

Test 3 Immersion Absorption Test

After curing, the immersion absorption test specimens were weighed and measured. They were then placed on a metal lattice raised 1 inch above the bottom of an 8 inch deep metal tank. Water was added and throughout the test was kept at a level about 1 inch above the tops of the specimens.

Specimens were weighed daily and a daily temperature record was kept. After 14 days of soaking the specimens were tested in unconfined compression. The final volatile content of the specimens was then determined.

The above procedure was employed for both the MC-3 and the portland cement stabilized specimens.

Soaking of test specimens was carried out in a manner identical to that outlined on page 456 ASTM (1956) except that larger specimens were used and the specimens also had access to water from the bottom. Temperatures in the immersion tank ranged from 16 C to 21 C due to variations in room temperature.

The 14 day immersion time was decided on because at immersion times less than this, the difference in absorbed water in specimens of different asphalt content was quite small. The samples were not completely saturated and were still absorbing water after 14 days.

Considerable slaking of sand was evident from the 3 per cent asphalt specimens and to a lesser extent from the 5 per cent and 7 per cent specimens. Slaking was increased by the necessity of handling the specimens daily for weight determinations. This loss of sand from the specimens, if its amount could have been determined, would have increased the difference in percentage of voids filled with water for the three mixes.

There were day to day variations in the amount of

water absorbed by the specimens and this appeared to be consistent throughout all specimens. This may have been due to day to day fluctuations in water temperature.

The discontinuity in Figure 5 occuring after day 3 for the 7 per cent MC-3 specimens and after day 4 for the 3 and 5 per cent MC-3 specimens was caused by changing the soaking water completely. Endersby, Griffen and Sommer (1947) found that water previously used for soaking specimens was more destructive than fresh tap water when unconfined compressive strength was used as the criterion. A crushed granite and MC-2 was used in their testing.

Test 4 Heating and Cooling Test

The cured specimens were weighed and measured and 'Demec' points for length change measurements were then attached. 'Demec' points are metal discs, ¼ inch in diameter by 1/16 inch thick with an indentation in the center. (A detailed description of the method of attaching the points is given in Appendix C). The specimens were re-weighed and the reference length was carefully obtained. The specimens were then subjected to 10 cycles of heating and cooling. Each cycle consisted of 12 hours of heating at about 150 F in a gravity oven and 12 hours of cooling at room temperature in the moisture room. The length was measured to 0.0005 inches after each phase of every cycle. After 10 cycles of heating and cooling the specimens were re-weighed with the measuring points attached. The points were removed

and the specimens were tested in unconfined compression.

The above procedure was employed with both MC-3 and portland cement stabilized specimens.

This test was intended to determine the effect of temperature changes on the specimens. Measuring points, were added to the specimens and length changes recorded to find out if measurable length changes occurred due to the temperature differential.

Measurement of length changes are not complete for all specimens since the method of measuring was changed near the start of the cycling and variations prior to this are not considered accurate enough to warrant their inclusion.

Test 5 Closed Freeze-Thaw Test

Cured specimens were weighed and measured. 'Demec' measuring points were then attached. The specimens were then weighed and the point to point lengths were measured. Specimens were then completely immersed in tap water at room temperature for 5 days, then weighed and measured. They were next subjected to 10 cycles of freezing and thawing, each cycle consisting of 12 hours of freezing in a deep-freeze at minus 15 C. to minus 23 C and 12 hours of thawing at room temperature in the moist room. Length measurements to 0.0005 inches were made after each phase of each cycle. After the tenth thawing phase the samples were weighed with points intact, the points were removed and the specimens were strength tested in the unconfined compression apparatus.

The above procedure was followed explicitly for the 18 MC-3 stabilized specimens. The 6 portland cement stabilized specimens were treated identically except that there was no soaking period prior to the freeze-thaw test. The soaking was omitted since, in the immersion test, very little water was absorbed in 14 days.

The primary intent of this test was to find the reduction in strength, caused by a number of cycles of freezing and thawing, on partly saturated samples of 3, 5, and 7 per cent MC-3 content, and 8 per cent cement content. The effectiveness of the stabilizing agents in reducing the effects of freezing and thawing could thus be determined.

The secondary intent of this test was to find if length changes during freezing and thawing were significant and if these could be accurately measured. The idea of using length change measurements in establishing a criteria for accepting or rejecting a mix came from Packard (1962). In Packard's work with portland cement stabilized specimens, length change was found to show promise as a suitability criterion, however, few details were given of the test procedure employed.

The procedure adopted was partly satisfactory but the specimens lost moisture during the test and the length changes decreased until they became very small. In further work the samples should be protected from moisture loss during the test, or immersed in water during the thaw period.

Test 6 Open Freeze-Thaw Test

The open freeze-thaw employed was similar to the modified British freeze-thaw test, except that a reservoir allowing 18 specimens to be handled simultaneously was used. (Watt, 1961) The water level in the box was kept at a level of $\frac{1}{4}$ inch above the bottom of the specimens during the test.

Specimens were subjected to 10 cycles of freezing and thawing. Each cycle consisted of 12 hours of freezing and 12 hours of thawing, with the specimen bottoms in continous contact with water. Freezing temperatures ranged from minus 16 to minus 20 C and thawing was at room temperature, that is 23 to 24.5 C, in the moist room.

At the conclusion of the tenth thawing cycle the specimens were extruded from the polyethylene tubing, weighed and measured, and tested in unconfined compression.

Watt used a temperature of minus 15 C for his freezing phase, whereas a temperature of minus 20 C was employed in this investigation. Watt states that no ice formed in the water reservoir after 12 hours of freezing. In this investigation, using the same apparatus , there was always at least 1 inch of ice around the inside of the box after 12 hours of freezing. There was however, only a thin film at most on the specimen bottoms, and it was felt that the frost line had penetrated through the specimens before this film formed.

Freezing in the reservoir may have been because only one box was in the freezer, with a large air space on all

sides, while in [illegible]'s work boxes were packed closely together and the temperature beside the boxes may have been higher. No ice lensing or frost heaving was evident in either the [illegible]C-3 or the portland cement stabilized specimens.

Since there was no evident vertical expansion, or deterioration of the specimens apparent after 10 cycles of freezing and thawing, the test was terminated then. No measurable lateral expansion of the specimens had occurred after the freeze=thaw test and extrusion from the molds was easily done by hand.

The British freeze-thaw test (Ref.5) and the Iowa modification of the British test (Davidson and Bruns 1960) using 14 cycles of freezing and thawing were used as models in conducting the Open Freeze-Thaw Test. The above methods utilize the unconfined compression test at strain rates of 0.05 and 0.10 inches per minute respectively as a means of comparing test specimens to control specimens. The British test uses a 4 inch by 2 inch cylindrical specimen in a vacuum bottle while the Iowa modification uses a 2 inch by 2 inch specimen. The procedure employed in this investigation differed in several ways from that outlined in the Iowa and British tests.

The curing period of 7 days with no evaporation was obviously not intended for use with cutback asphalt stabilized material. The specimen top was asphalt coated to prevent evaporation and 24 hour soaking periods after curing were used

in the British and Iowa methods, while neither were employed here. In this investigation a freezing temperature of minus 20 C and a thawing temperature of 23 C were used, while both of the guide methods used temperatures of minus 5 C and 25 C. In addition, freeze and thaw phases of 12 hours each were used, compared to 16 hours freezing and 8 hours thawing in the Iowa and British methods. A vacuum bottle used in the guide methods was replaced by a box containing 18 specimens, constructed by Watt (1961). The tops of the specimens were not coated in this investigation, since the specimens had been cured for 5 days as a drying measure, no soaking was employed, hence there was no reason to prevent moisture loss from the specimens. The more severe freezing temperature was used because of the shorter freezing time employed. The 12 hour freezing and thawing phases were adopted as a convenience in the investigation, and to assure that the specimens were completely thawed before re-freezing.

Test 7 Closed Freeze-Thaw At Constant Water Content

After 5 days of curing the test specimens were weighed, then soaked for 5 days. Specimen diameter, length, and weight were then determined and the specimens were sealed in polyethylene bags. Ten cycles of freeze-thaw were then performed as for Test 5, with freezing at minus 15 to minus 20 C, and thawing at 23 to 24.5 C. After the tenth thawing cycle, specimen height and diameter were measured, and the specimens were tested in unconfined compression.

This test was added at the conclusion of the testing program in hopes of overcoming some of the deficiencies mentioned for the similar freeze-thaw test in which length changes were measured (Test 5). It was felt that this test would be more severe than Test 5, wherein specimens gradually dried out during the freeze-thaw cycles. No length change measurements were made on the specimens in this test.

Triaxial specimens, 2.8 by 6.5 inches, were also subjected to cyclic freezing and thawing. This durability test was identical to Test 7, performed on the 2 inch by 2 inch specimens except that the soaking time was 7 days instead of 5 and after the 10th thawing cycle the specimens were tested in triaxial compression as described on page 37.

Unconfined Compression Test Procedure

This test was conducted with the Soiltest machine shown in Plate 2, using a 600 pound capacity proving ring for the MC-3 stabilized specimens and a 2000 pound ring for the portland cement stabilized specimens.

Strain was controlled in the testing to a rate of approximately 0.03 inches per minute. (Strain control is referred to as approximate since, while the rate of the strain dial movement was accurately controlled, the true strain at any instant was equal to the strain dial reading minus the proving ring deflection as measured by the load dial).

The load dial was first zeroed and the strain dial was approximately zeroed. A 2-¼ inch by 1/8 inch brass plate was

UNCONFINED COMPRESSION TEST APPARATUS

PLATE 2.

placed on the platform under the specimen. The load leveling device was then placed between the specimen and the proving ring. The platform was then raised by the hand crank until firm contact was made, as indicated on the proving ring load dial. The proving ring pointer was then returned to zero by carefully cranking off the load. The strain dial was then set at zero and compression was begun.

Straining of the specimen was accomplished by raising the platform at a rate of 0.01 inches per 20 seconds (0.03 inches per minute) using the hand crank and continuously checking a timer and the strain dial. Failure load was considered to be reached when the specimen strained without a significant load increase. Load versus deformation readings were taken for at least one specimen from each group of 6.

After maximum load was reached the load was cranked off, the specimen examined, and then discarded or weighed for a final moisture content determination.

The usual conical type of fracture found in asphalt and cement stabilized materials when tested in compression (Plate 1a) is closely related to the friction on the ends. With no end friction a longitudinal splitting could be expected. The length to diameter ratio of specimens for compression testing should be about 2. The strength values obtained for specimens with length to diameter ratios of 1, as employed in this investigation, would be higher than strengths of specimens of the same material with a higher length to diameter ratio. (Morris-

son and Ford, 1955).

The strain rate of 0.03 inches per minute was selected for its convenience in the strain controlled test where the strain was applied manually. Rice (1949) in similar tests on the same size specimens used a strain rate of 0.0105 inches per minute, while Watt (1961) using 2 inch by 2 inch lime stabilized specimens used a rate of .087 inches per minute.

In soil stabilization neither the compressive strength or the modulus of elasticity are normally used for design, but are used for comparison purposes. Therefore it is important to have identical test conditions where a comparison is to be made.

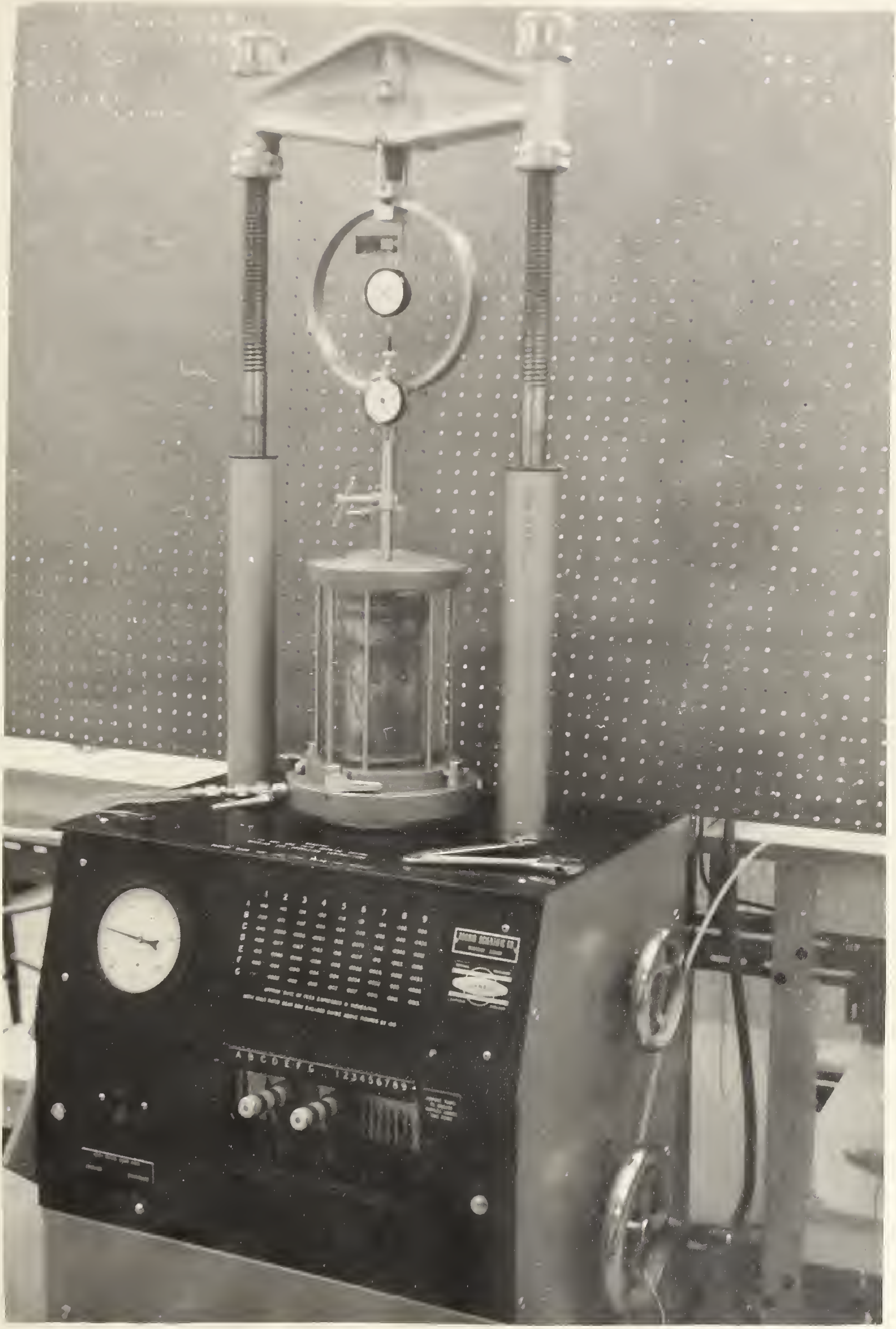
Triaxial Compression Test Procedure

Twenty seven specimens were prepared for testing in the Farnell machine shown in Plate 3, after 10 cycles of freezing and thawing. Three of each group of 9 specimens were tested at zero lateral pressure, i.e. unconfined, 3 at twenty psi. lateral pressure, and 3 at 30 psi, A strain rate of .01 inches per minute was employed. The detailed test procedure was as follows:

1. The triaxial cell, base, and membrane were thorough-ly washed.

2. The membrane was rolled up and placed around the base, then fastened securely with 'O' rings and elastic.

3. The specimen was placed on the base, the membrane rolled up around it, and the head placed on the specimen to



TRIAXIAL COMPRESSION TEST APPARATUS

PLATE 3.

which the membrane was stretched. The levelling ball was placed on top of the specimen and the cell fastened in place.

4. The cell was then filled with water through the top until almost full and a small amount of oil added on top. The piston was held in against the water, and water was drained from the bottom until the piston was seated.

5. The triaxial cell was now placed under the proving ring in the testing machine and the lateral pressure was adjusted to the required value.

6. The loading piston was then seated on the specimen and the strain and load dials zeroed.

7. The speed control on the testing machine was set at the rate of 0.01 inches per minute and the loading begun.

8. Strain and load dial readings were taken every .05 per cent strain for the first 0.2 per cent, every 0.1 per cent up to 1 per cent, and every 0.2 per cent until maximum stress was reached.

9. The load was released and the sample removed from the apparatus.

After the load had been determined from the proving ring calibration charts, a rough load versus strain plot was made for each specimen. This provided a check on the seating of the piston on the specimen since this curve must pass through the origin. If the curve did not pass through the origin it was assumed that part of the load was utilized to seat the piston. When the curve began to rise on an approximately

straight line the load was being transmitted to the specimen. This line was extended to intersect the strain axis, and the intercept was taken as the true zero strain. All readings at strains lower than this value were ignored, and all higher strain readings were corrected by subtracting this value. No correction was applied to load dial readings, and only two of the specimens tested required a correction to be applied to the strain.

Analysis of Data

The effects of the durability tests were judged by the unconfined compressive strengths of specimens subjected to them. Failure loads were obtained from calibration curves for the various proving rings employed. In calculating the area at maximum load, the specimens were assumed to deform at constant volume. The formula $A_c = \frac{A_o}{1-e}$ was then used to determine the area at failure wherein

A_c = area at maximum load - sq. in.

A_o = initial area - sq. in.

e = unit strain

No correction was applied to compressive strengths for the height to diameter ratio employed.

Calculations of voids in the specimens were made assuming a constant volume of 103.7 cubic centimeters, on the basis of 2.000 inch diameter by 2.015 inch height. The measured diameter of the mold was used and many specimen height measurements showed that the height dimension used was reasonable. A table

was set up showing the weight of sand, water and MC-3 present for any compacted weight of specimens, based on design water and MC-3 percentages. Then assuming 79.58 per cent residue from the MC-3 (the average value from two samples after 15 days at 110 C) the residue for each weight of MC-3 was calculated.

The weight of sand plus residual asphalt in each specimen was calculated from the initial wet weight and the volatile content on a percent of sand plus residual asphalt basis. The weight of residue as determined above was subtracted, giving the weight of dry sand in each specimen. Volume of sand was computed using a specific gravity of 2.66, and the volume of voids was obtained by subtracting this value from the specimen volume. The specific gravity of the asphalt was taken as 1.00 (Asphalt Handbook 1960)

The volume of water in a specimen at any time was the weight, minus the weight of sand plus residual asphalt. In order to simplify calculations, void calculations were made for each group of specimens rather than individual specimens and the result was therefore the average for that group.

Where length change measurements were made as in the heat-cool and freeze-thaw tests, the initial separation of reference points for each specimen was used as its length. All later measurements were compared to this standard. Measured length changes were averaged and plotted against number of cycles.

Triaxially tested specimens were corrected for area in the same manner as the smaller unconfined test specimens.

From the triaxial test data, Mohr rupture lines and

stress-strain plots were drawn and compared with results from another investigation with the same material.

CHAPTER V TEST RESULTS AND DISCUSSION

Preliminary Testing

Figures 1 and 2 and tables VII and VIII (Appendix B) present a summary of the results of preliminary testing.

Figure 1 is a plot of unconfined compressive strength against curing time at 100F. From this curve the curing time of 120 hours, used throughout the test, was selected. Even though this curve is not obtained from the highest asphalt content employed, the cured weights of specimens with 7 percent MC-3 indicated little volatiles remaining. Rice (1949), at Purdue University used a curing time of 120 hours at a temperature of 110 F with the same specimen size.

Any curing conditions in laboratory work should be selected after considering the environmental conditions in the locality and the physical conditions in the pavement structure. Ideally therefore, the samples should be exposed to the atmosphere on one face only, and on this face be exposed to varying, or average, conditions of wind, temperature, and humidity. An estimate would have to be made of the state of curing allowed before the base would be subjected to loads. Laboratory duplication of the above conditions would be possible, however, the time to develop the required strength would be considerably

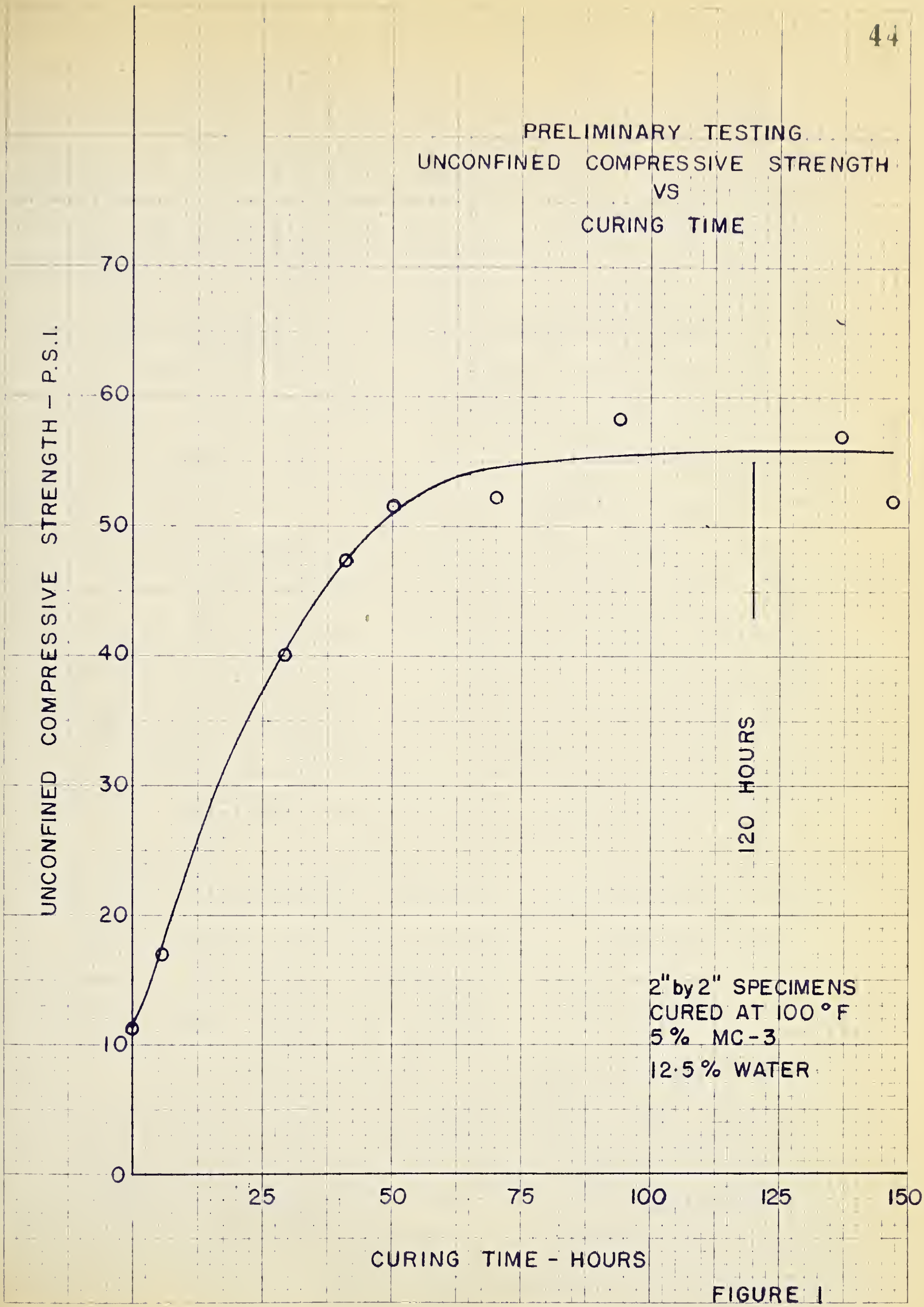

PRELIMINARY TESTING
UNCONFINED COMPRESSIVE STRENGTH
VS
CURING TIME
UNCONFINED COMPRESSIVE STRENGTH - P.S.I.
0
10
20
30
40
50
60
70
25
50
75
100
125
150
CURING TIME - HOURS
120 HOURS
2" by 2" SPECIMENS
CURED AT 100 °F
5 % MC-3
12·5 % WATER

FIGURE I

longer, and the degree of curing before testing would still have to be estimated. For these reasons, and to save time, laboratory curing is usually accelerated as compared to natural conditions.

A plot of unconfined compressive strength versus percent MC-3 in Figure 2 summarizes the results of the preliminary testing for the effect of a freeze-thaw test on cured and uncured specimens, composed of varying percentages of liquid asphalt. The abrupt drop in strength of 5 percent MC-3 specimens, as compared to 3 percent MC-3 specimens, created suspicion as to the validity of the results. Since a 2000 pound proving ring was used for some of the first specimens tested, and later a 600 pound ring was obtained, an error in calibration was suspected. However, the results of the main testing program showed the same trend, as is shown in Figure 3. The strengths of the cured and uncured specimens showed a completely different trend. The trend shown by the cured specimens was believed to be more realistic and the loss in strength after freezing and thawing was greater. Therefore the testing of uncured specimens was excluded from the remainder of the program.

The average sand densities in the specimens with from 1 to 11 percent MC-3 showed no significant variation with different asphalt contents. The average dry density was 102.0 pounds per cubic foot., and the range of average

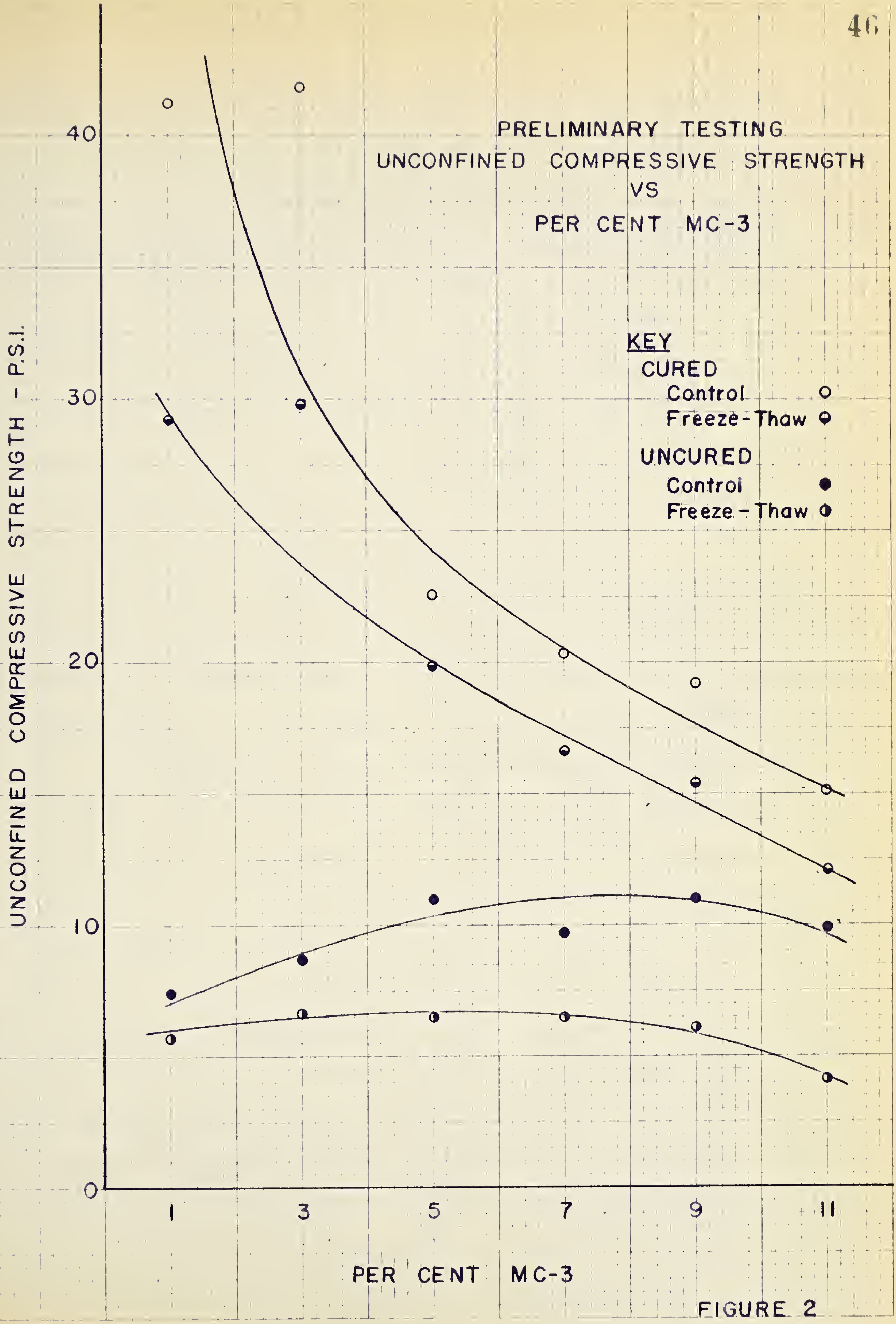

PRELIMINARY TESTING
UNCONFINED COMPRESSIVE STRENGTH
VS
PER CENT MC-3
KEY
CURED
Control
Freeze-Thaw
UNCURED
Control
Freeze-Thaw
UNCONFINED COMPRESSIVE STRENGTH - P.S.I.
40
30
20
10
0
1
3
5
7
9
11
PER CENT MC-3

FIGURE 2

densities was from 101.8 at 1, 5 , and 11 per cent MC-3, to 102.4 pounds per cubic foot at 9 per cent MC-3.

Results of Durability Tests

Capillary and Immersion Absorption Tests (Tests 2 and 3)

Figure 3 summarizes graphically the results of unconfined compression tests on specimens subjected to immersion for 14 days (Test 3), and to capillary absorption (Test 2) for 14 days, compared to control test results. Each point on all graphs in this section is an average for 6 specimens. The maximum variation from the average strength was less than 10 per cent of the average strength in all cases.

Figures 4 and 5 are plots of per cent total voids filled with water plus residual asphalt, and voids filled with water only, against time of total immersion at room temperature. The method of calculating voids was described in Chapter IV.

The faults of the capillary absorption test as applied to this material have been described in Chapter IV. A decrease in strength with increasing asphalt content similar to that observed in the control test (see Figure 3) was expected, considering that the only apparent effect of the test was to increase the degree of saturation of the bottom quarter inch of the specimen, which was immersed. The loss of strength caused by this test was surprisingly large, being approximately 75 per cent of cured specimen strength for each asphalt content. The immersion test, on the other hand, caused a loss in strength of 82

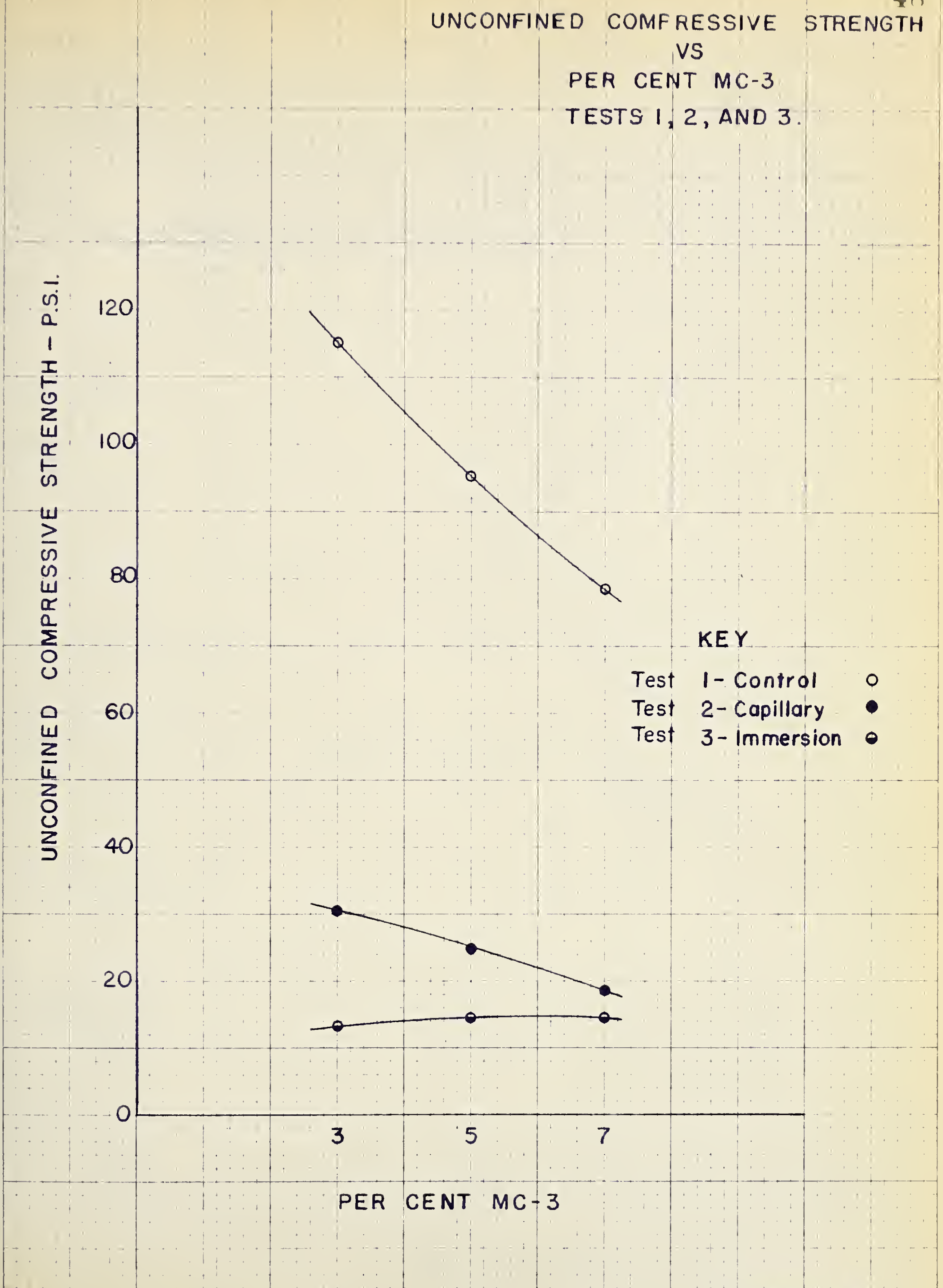

UNCONFINED COMPRESSIVE STRENGTH
VS
PER CENT MC-3
TESTS 1, 2, AND 3.
UNCONFINED COMPRESSIVE STRENGTH - P.S.I.
120
100
80
60
40
20
0
3
5
7
PER CENT MC-3
KEY
Test 1- Control
Test 2- Capillary
Test 3- Immersion

FIGURE 3

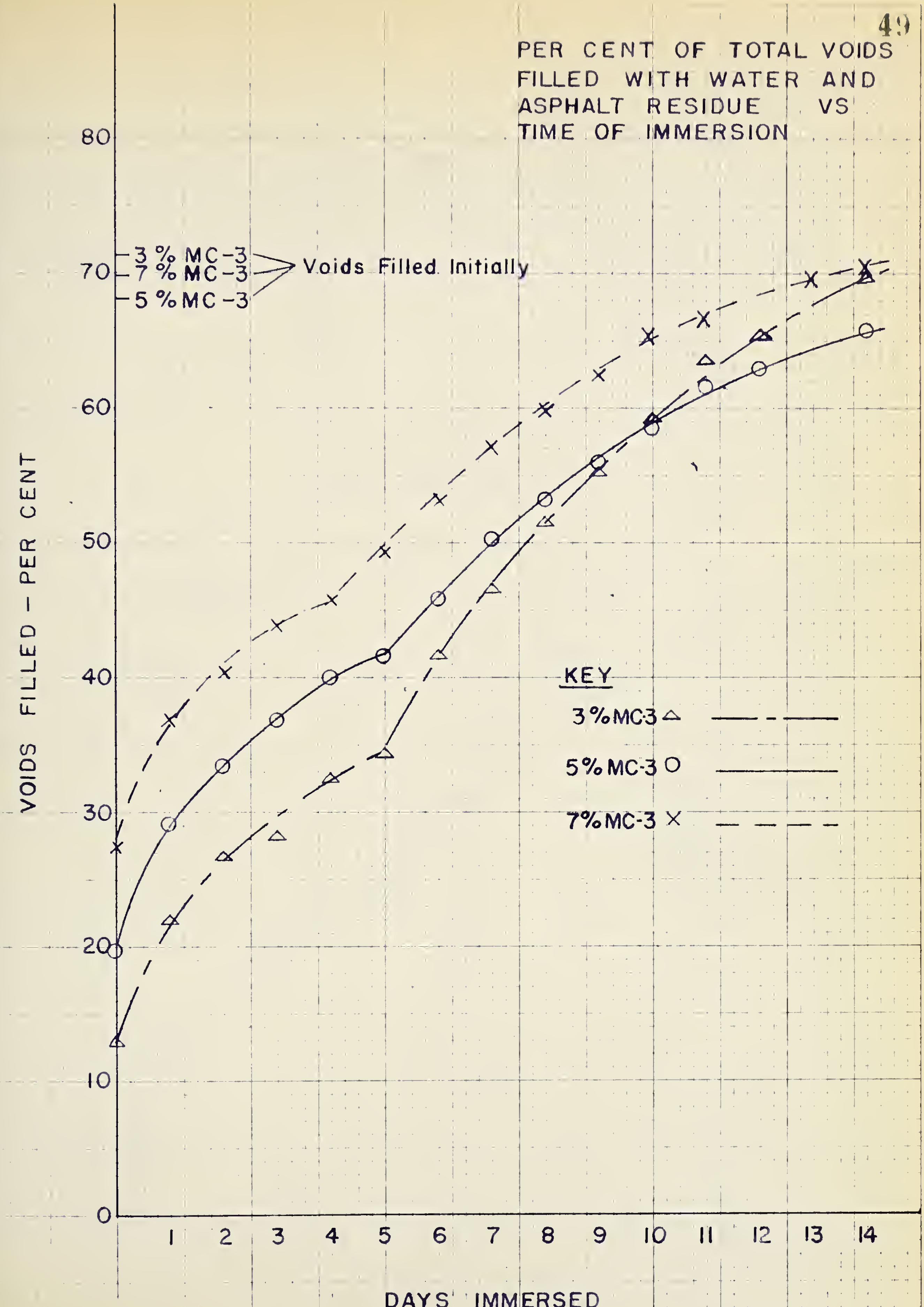

PER CENT OF TOTAL VOIDS FILLED WITH WATER AND ASPHALT RESIDUE VS TIME OF IMMERSION
3 % MC-3
7 % MC-3
5 % MC-3
Voids Filled Initially
VOIDS FILLED - PER CENT
0
10
20
30
40
50
60
70
80
1
2
3
4
5
6
7
8
9
10
11
12
13
14
DAYS IMMERSED
KEY
3%MC-3
5%MC-3
7%MC-3

FIGURE 4

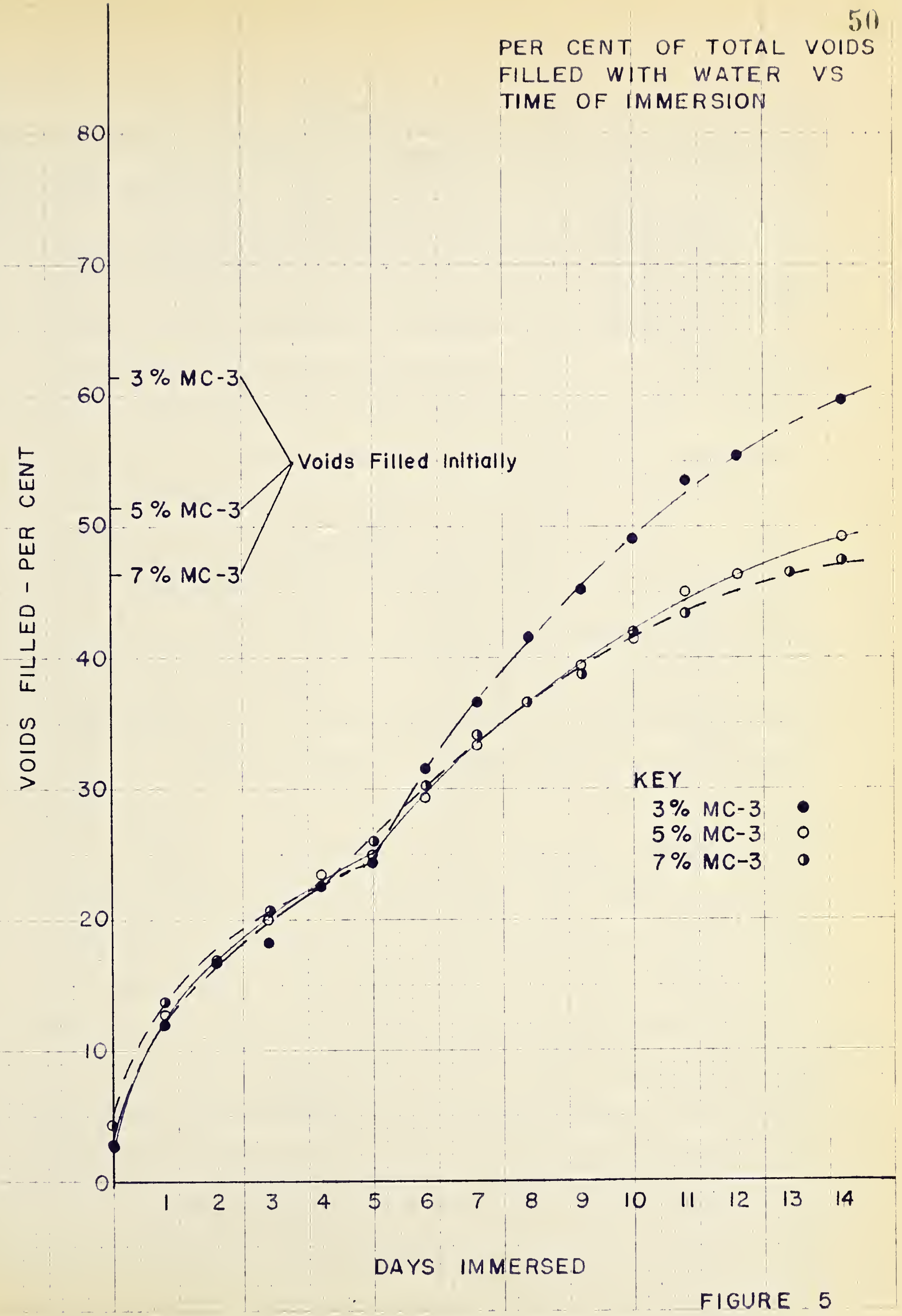

PER CENT OF TOTAL VOIDS FILLED WITH WATER VS TIME OF IMMERSION
VOIDS FILLED - PER CENT
0
10
20
30
40
50
60
70
80
3 % MC-3
5 % MC-3
7 % MC-3
Voids Filled Initially
KEY
3 % MC-3
5 % MC-3
7 % MC-3
1
2
3
4
5
6
7
8
9
10
11
12
13
14
DAYS IMMERSED

FIGURE 5

to 89 per cent. However, specimens subjected to the immersion test absorbed 5 to 6 times as much water as capillary test specimens.

The explanation for the seemingly low strength of capillary absorption specimens compared to immersed specimens may lie in the physical conditions prevailing in the specimens. Neglecting diametrical differences in saturation, the immersed specimens would have a uniform degree of saturation from top to bottom. Capillary test specimens though, consisted of two zones, the bottom 1/8 of the specimen, with approximately the same saturation as immersed specimens and the remainder of the specimen, in its original cured state. Therefore in the capillary test specimens, the observed (by maximum load) failure may have occurred in the more saturated part of the specimen with the remaining drier part having only a limited effect on strength.

After 14 days of immersion, specimens with 3, 5 and 7 per cent MC-3 had almost identical, and very low strengths. The percentage of voids filled, after 14 days, was about equal to or lower than the initial degrees of saturation of the specimens as shown in Figures 4 and 5. The test sand stabilized with any of these asphalt contents had very low resistance to the effect of water and could not be judged satisfactory for field use from this test.

A test similar to the procedure followed here is

sometimes used as a measure of a mix's resistance to stripping (Pauls and Goode 1947). With this test it has been found that penetration grade asphalt specimens retain the highest per cent of dry strength after immersion, there is a great strength loss on immersion with rapid curing asphalts, and an even greater loss with medium curing asphalts.

The Heating and Cooling Test (Test 4)

Unconfined compressive strengths after 10 cycles of heating to 150 F. and cooling to room temperature (approximately 70 F.) are shown compared to control specimen strengths in Figure 6. The results of length change measurements are plotted in Figure 7.

Considering the range of length changes over 10 days, specimens with 3, 5 and 7 per cent MC-3 showed identical characteristics. Normally, the specimens swelled with heating and shrank on cooling. However, there are several variations from the normal in this plot. It appears that the accuracy of the measurement in length changes was not sufficient to show true changes in length. Therefore the length changes with heating and cooling as measured in this test were insignificant and no trend can be established from them.

Specimens made with 3 per cent MC-3 showed an average strength gain of 16 psi when tested in compression. In contrast, specimens with 5 and 7 per cent MC-3 lost 19 and

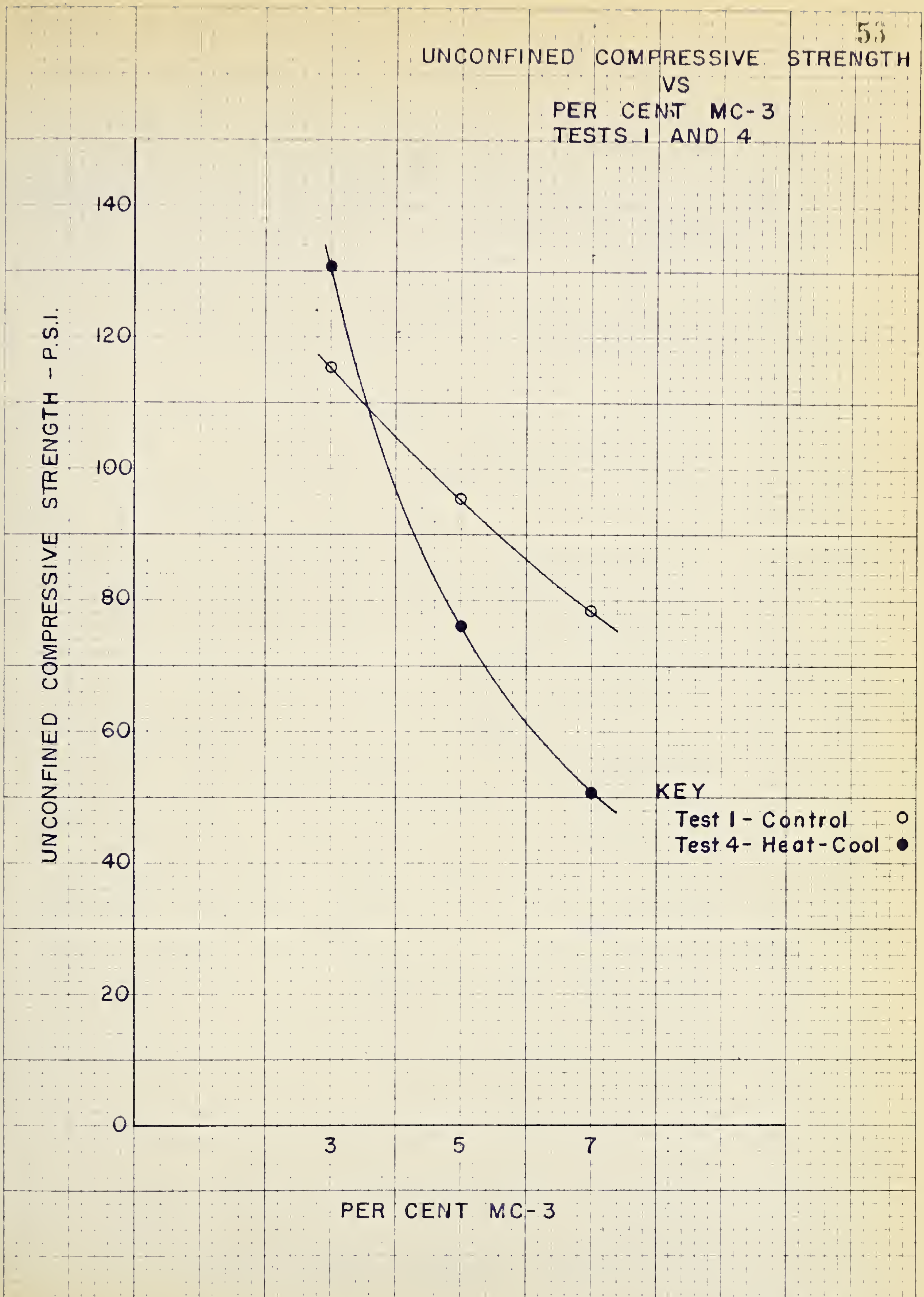
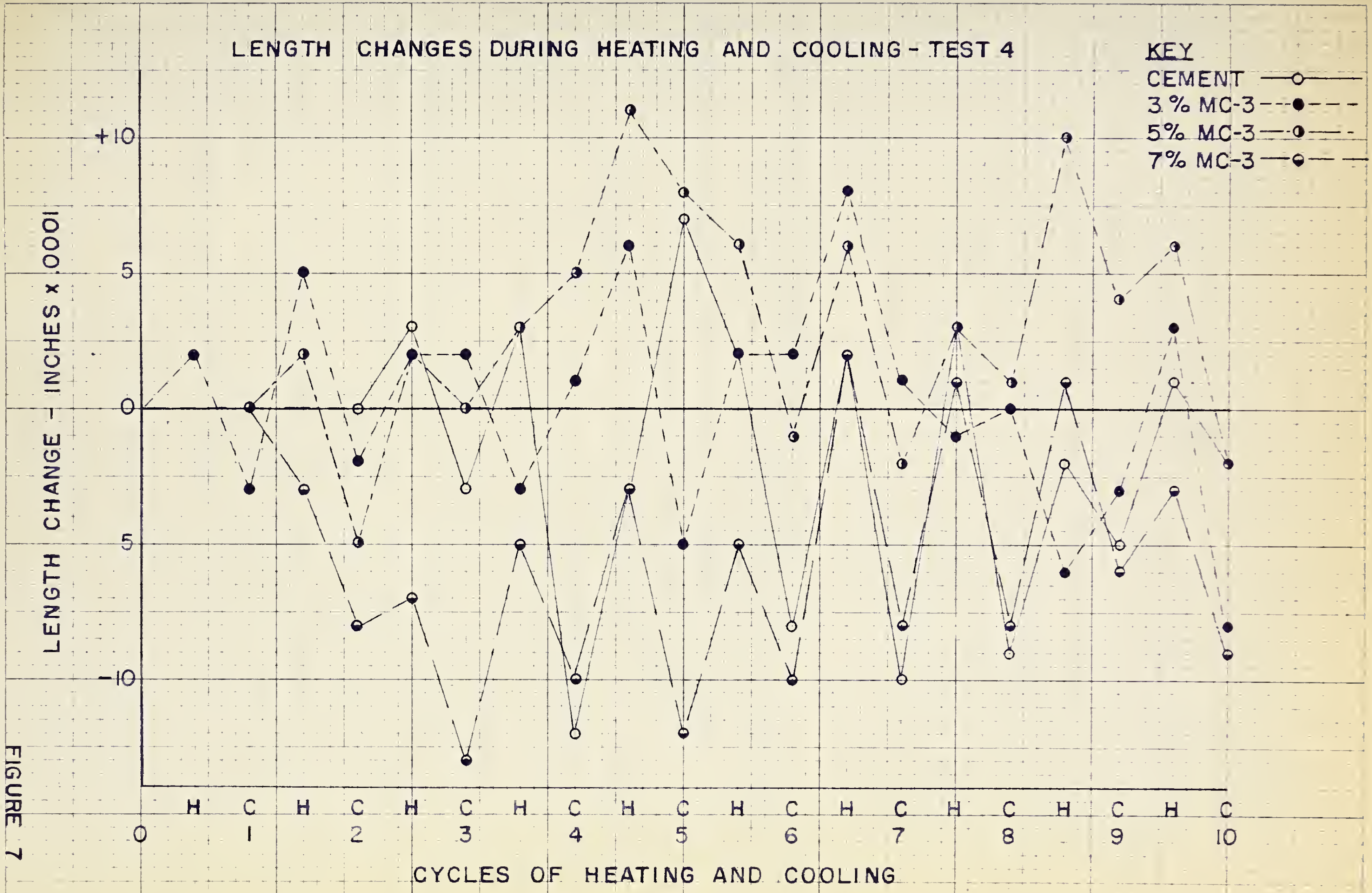

UNCONFINED COMPRESSIVE STRENGTH
VS
PER CENT MC-3
TESTS 1 AND 4
UNCONFINED COMPRESSIVE STRENGTH - P.S.I.
140
120
100
80
60
40
20
0
3
5
7
PER CENT MC-3
KEY
Test 1 - Control
Test 4 - Heat-Cool

FIGURE 6



LENGTH CHANGES DURING HEATING AND COOLING - TEST 4
KEY
CEMENT
3 % MC-3
5% MC-3
7% MC-3
LENGTH CHANGE - INCHES x .0001
+10
5
0
5
-10
H C H C H C H C H C H C H C H C H C H C
0 1 2 3 4 5 6 7 8 9 10
CYCLES OF HEATING AND COOLING

FIGURE 7

27 psi strength in compression respectively. All specimens were tested at the same temperature, therefore there was no difference in the viscosity of the residual asphalt. It must therefore be concluded that the decrease in strength with increasing asphalt content, evident from the strength of control specimens, was increased when specimens were subjected to heating and cooling. It is possible that capillary forces were introduced in the 3 per cent MC-3 specimens which could not develop in the 5 and 7 per cent specimens.

Freeze-Thaw Tests Numbers 5, 6 and 7

The unconfined compressive strengths of specimens subjected to the three freeze-thaw tests are shown compared to control specimen strengths in Figure 8.

Comparative strengths of Test 5 and Test 7 specimens indicate the effect of water on compressive strength. Before the freeze-thaw cycling began the specimens for both tests had been soaked for five days. Then Test 7 specimens were sealed in air-tight bags, preventing evaporation, but Test 5 specimens were unprotected during freezing and thawing. Therefore, during freezing and thawing, assuming the same volume after freezing and thawing as before, the per cent of total aggregate voids filled with water decreased as follows:

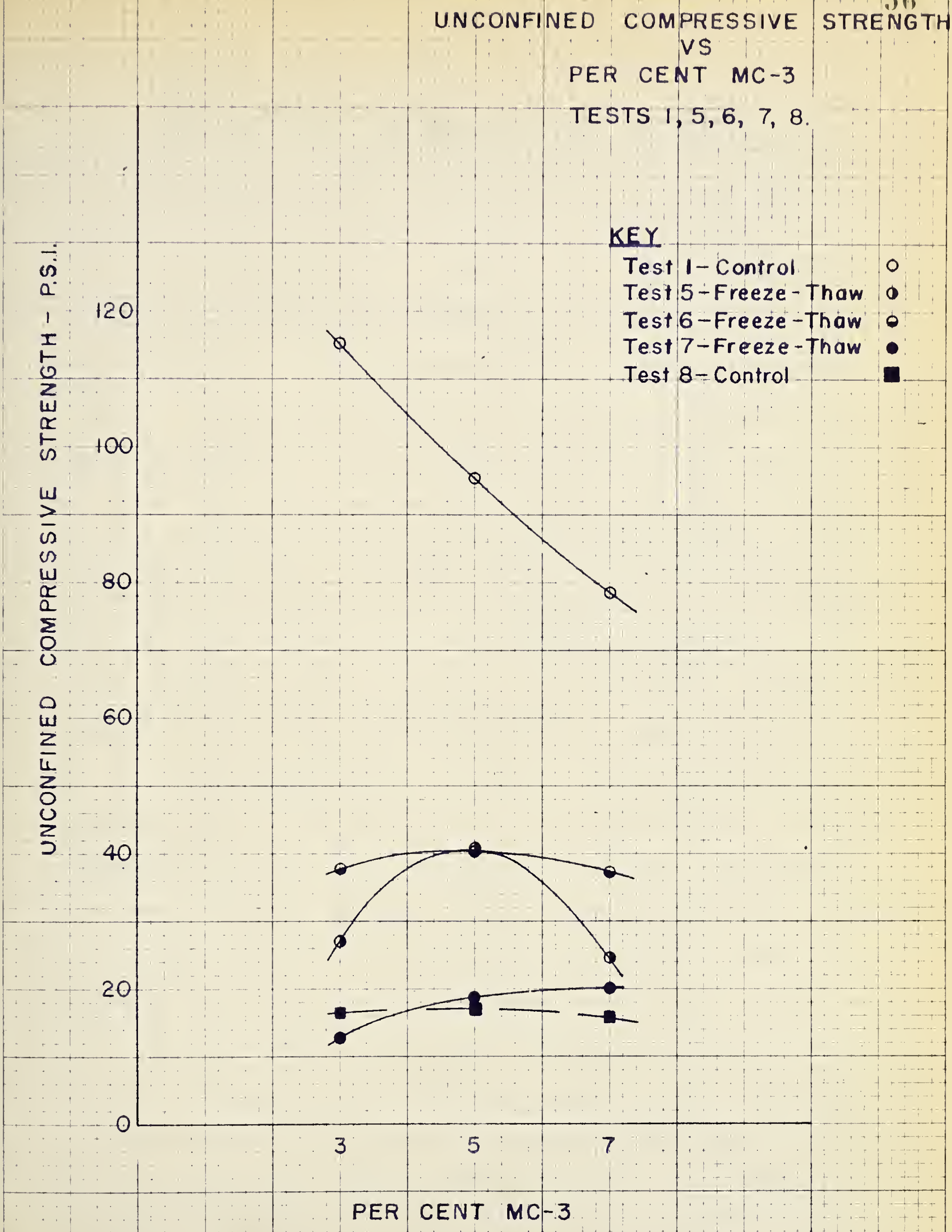

UNCONFINED COMPRESSIVE STRENGTH
VS
PER CENT MC-3
TESTS 1, 5, 6, 7, 8.
KEY
Test 1 – Control
Test 5 – Freeze - Thaw
Test 6 – Freeze - Thaw
Test 7 – Freeze - Thaw
Test 8 – Control
UNCONFINED COMPRESSIVE STRENGTH - P.S.I.
120
100
80
60
40
20
0
3
5
7
PER CENT MC-3

FIGURE 8

Test 5

Per Cent MC-3	% Water Filled Voids Before F/T	% Water Filled Voids After F/T
3	34.8	13.9
5	27.9	13.5
7	24.5	9.4

A peak strength is indicated in Figure 8 at 5 per cent MC-3 for Test 5. The original 5 per cent MC-3 samples used in this test were damaged by attempting to replace the length measurement points, which were not properly attached, and came loose when the specimens were soaked. The results reported are for 6 specimens, 551A to 556A, formed to replace the original series. The apparent optimum MC-3 content of 5 per cent for this test was suspected of being in error, since, when the specimens were formed, the liquid asphalt was noted to be stiffer than when previously used. It was later suspected that the level of asphalt in the storage barrel had dropped below the heating element, and the temperature may have been lower than indicated. The affect of this difference, if any, on unconfined compressive strength is unknown. Length change measurements in Test 5, shown in Figure 9, also indicate a difference in 5 per cent MC-3 specimens, as compared to 3 and 7 per cent specimens. The 3 per cent specimens showed average length increases as great as .007 inches in the early freeze-thaw cycles, while 7 per cent samples showed the same general trend, but a lesser amplitude. However, 5 per cent MC-3 specimens failed to

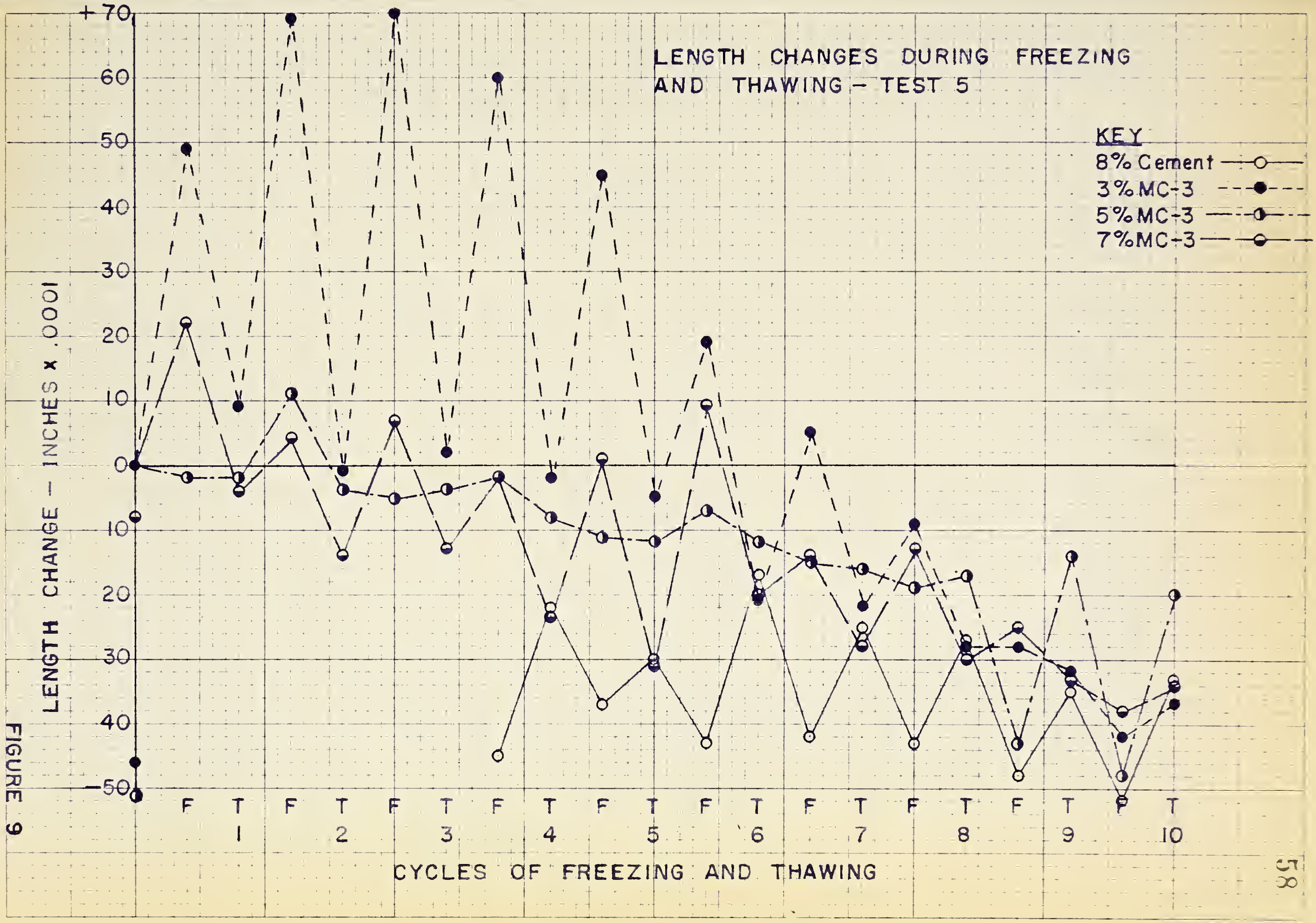

LENGTH CHANGES DURING FREEZING
AND THAWING – TEST 5
KEY
8% Cement
3% MC-3
5% MC-3
7% MC-3
LENGTH CHANGE – INCHES x .0001
+70
60
50
40
30
20
10
0
10
20
30
40
–50
F T F T F T F T F T F T F T F T F T F T
1 2 3 4 5 6 7 8 9 10
CYCLES OF FREEZING AND THAWING

FIGURE 9

follow the expansion contraction pattern established by the other two groups, but showed the same shrinking trend as the test progressed. This may have been due to the suspected difference in asphalt consistency when mixed, as previously noted.

The two lower curves in Figure 8 represent Test 7, a closed freeze-thaw test where no evaporation was permitted, and Test 8, a control test identical to Test 7 except that no freezing and thawing was carried out. The unconfined compressive strengths are so nearly identical for the two tests that it may be concluded that the freezing and thawing test had no detrimental effect on the strength. The unconfined compressive strengths from Test 3, 14 day immersion (Figure 3), are only slightly lower than the strengths of specimens subjected to only 5 days immersion, even though the percentage of voids filled with water is over 3 times as great (Tables III, IV, and V). It is therefore apparent that a high degree of saturation is not essential for a large strength loss.

The 'open' freeze-thaw test, Test 6, was a modification of British Standard 1924. There was no heaving or ice lensing evident in any specimens subjected to this test. The specimen bottoms, which were in contact with water for the 10 day duration of the test, showed deterioration varying inversely with the amount of asphalt used. The specimens with 3 per cent

asphalt were eroded significantly by the water, while the 5 per cent samples suffered to a lesser extent, and very little deterioration was obvious in the 7 per cent MC-3 specimens. This test was the only durability test employed that showed any visual evidence of the superiority of the higher asphalt content. Even then, the strength of 7 per cent MC-3 specimens was slightly lower than strengths of 3 and 5 per cent MC-3 specimens. In this test, a lower proportion of voids were filled with water in the 5 per cent MC-3 than in the 7 per cent MC-3 specimens. This may account for the slight peak in the strength versus asphalt content curve.

Control Tests

The two control tests conducted were designated Tests 1 and 8. Test 1 specimens were tested in compression after 5 days of curing. In addition to curing, Test 8 specimens were immersed in water at room temperature for 5 days for comparison with the freeze-thaw tests.

The very severe effect of water immersion on the compressive strengths are shown. This is further emphasized when the strengths of Test 1 specimens are compared to that of the control specimens in Figure 2, which were soaked only 24 hours prior to testing.

When specimens are immersed with access to water over the entire surface, wetting proceeds from the outside toward the centre. In the preliminary testing visual examination of

failed specimens soaked only one day showed a thin, wetted shell on the outside. Since the strength of these specimens was much lower than that of Test 1 specimens which had not been soaked, it was apparent that the weaker portion of the specimens controlled the compressive strength.

A pronounced decrease in strength with increasing asphalt content was found in Test 1, which substantiated the trend established in the preliminary testing. This trend was hardly noticeable after five days immersion, and was not apparent after 14 days.

In sand specimens with little or no asphalt, drying caused the water menisci between the sand grains to decrease in radius and retreat toward the intergranular contact. Therefore, the tension in the pore water increased with drying, thereby increasing the intergranular compressive forces. The increase in intergranular compression had a similar effect as increasing the all around pressure in a triaxial test and therefore the axial load necessary to cause failure was greater than in specimens without this effect. The strength decreased with increasing asphalt content; first, since evaporation was more difficult with higher asphalt contents and there was therefore less pore water tension produced, and second, since the asphalt probably acted as a lubricant to some extent, rather than as a cementing agent. Over a longer time the asphalt may have hardened and functioned as a cement,

SUMMARY OF RESULTS

TABLE III

Sand with 3% MC-3

Test	1 Control	2 14 day capillary absorption	3 14 day immersion absorption	4 Heat-Cool	5 Closed freeze-thaw	6 Open freeze-thaw	7 Closed freeze-thaw	8 Control soaked 5 days
Voids - per cent of aggregate volume	62.6	62.7	63.0	62.7	62.9	62.9	62.8	63.4
Voids filled with asphalt cement - per cent	10.1	10.1	10.0	10.1	10.1	10.1	10.1	10.0
Voids filled with water when tested - per cent	3.8	12.4	59.7	1.0	13.9	14.4	36.3	16.5
Bulk density after curing. lbs. per cu. ft.	105.5	105.4	105.2	105.1	105.3	105.0	105.2	104.9
Average compressive strength (psi)	115.2	30.2	13.2	130.7	27.0	37.8	12.6	16.5

SUMMARY OF RESULTS

TABLE IV

Sand with 5 % MC-3

Test	1 Control	2 14 day capillary absorption	3 14 day immersion absorption	4 Heat-Cool	5 Closed freeze-thaw	6 Open freeze-thaw	7 Closed freeze-freeze thaw	8 Control soaked 5 days
Voids - per cent of aggregate volume	62.9	62.2	62.6	62.3	62.3	62.9	62.2	62.8
Voids filled with asphalt cement - per cent	16.8	17.0	16.6	16.6	16.9	16.8	17.0	16.8
Voids filled with water when tested - per cent	2.6	13.3	49.1	2.3	13.5	13.4	34.1	17.4
Bulk density after during. lbs. per cu. ft.	106.8	107.6	106.8	106.6	107.3	106.4	107.4	107.1
Average compressive strength (psi)	95.5	24.7	14.2	76.0	40.4	40.0	18.8	17.0

SUMMARY OF RESULTS

TABLE V

Sand with 7 % MC-3

Test	1 Control	2 14 day capillary absorption	3 14 day immersion absorption	4 Heat-Cool	5 Closed freeze-thaw	6 Open freeze-thaw	7 Closed freeze-thaw	8 Control soaked 5 days
Voids - per cent of aggregate volume	64.0	62.3	63.2	63.7	63.3	64.2	63.2	61.4
Voids filled with asphalt cement - per cent	23.0	23.6	23.2	23.2	23.1	23.0	23.3	24.1
Voids filled with water when tested - per cent	3.0	13.6	47.4	2.7	9.4	16.0	31.6	17.3
Bulk density after during. lbs. per cu. ft.	107.8	109.0	108.3	108.3	108.3	107.8	108.4	107.6
Average compressive strength (psi)	78.2	18.3	14.2	50.5	24.4	37.2	20.1	15.7

but investigation of this phenomenon was beyond the scope of this program.

Tests on Sand With [illegible] Per Cent Portland Cement

The average unconfined compressive strengths of 2 inch by 2 inch specimens after the durability tests are as shown in Table VI.

The modified British, or open freeze-thaw test, was the only test to result in a substantial decrease in strength compared to control specimen strength. The 87 per cent of control strength remaining after freezing and thawing is still well above the minimum of 75 per cent allowed by the British test, or the minimum of 80 per cent allowed in the Iowa modification of the British test. Packard, (1962) found that the strength of acceptable freeze-thaw test specimens was always greater than that of 7 day cured specimens. This strength gain was attributed to additional curing during the thaw phases.

Specimens subjected to 10 cycles of heating to 150F. and cooling to room temperature had only about 2 per cent water remaining at the conclusion of the test, as compared to an initial moisture content of 16 per cent* The test did not adversely affect the strength of specimens, though, as the final strength was 10 per cent greater than the strength of specimens cured under a condition of saturation humidity. This greater strength may have been due to either the compressive intergranular forces produced by the water meniscii

* The 16 per cent value was the water added initially, while the 2 per cent value is an estimate from the decrease in specimen weight.

SUMMARY OF RESULTS*

TABLE VI

McGinn Sand With 8% Portland Cement

Test	Dry Unit Weight (lbs/cu.ft.	Compressive Strength (psi)	Per cent Control Strength**
1 Control 7 day cure	103.9	273.5	
1-A Control 17 day cure	104.1	323.2	100
3 Immersion 14 day	103.6	314.0	
4 Heating and Cooling	103.6	356.7	110.5
5 Closed Freeze-thaw	104.0	418.5	129.6
6 Open Freeze-thaw	103.9	280.5	86.8

* Each entry in the table is the average for 6 specimens.

** Strengths compared to 17 day moist cured strength.

as previously proposed for sand-asphalt specimens, or, as proposed by Packard (1962), to accelerated strength gain caused by the 150 F. heating temperature.

Average length changes during the heat-cool and freeze-thaw tests are shown in Figures 7 and 9 respectively. In both cases, length measurements were not made from the start of the test, so no conclusions can be drawn as to overall length changes during the tests.

In the heating and cooling test a general pattern of expansion on heating and contraction on cooling is evident. Since the scale of the plot is greatly exaggerated compared to the precision of the measurements, a comparison of the behaviour of sand cement and sand-asphalt specimens is not warranted.

Measured length changes of sand-portland cement specimens in the freeze-thaw test were greater than in the heat-cool test. They were small compared to the most severe changes measured in sand-asphalt specimens though, and, through several cycles, showed contraction when frozen instead of the expansion indicated by asphalt specimens. In tests conducted with various soils and cement contents, Packard (1962) found that acceptable specimens shrank on freezing and expanded on thawing, while badly deteriorated specimens expanded upon freezing. In addition, soil type was found to be the principle factor affecting the magnitude of length changes.

Triaxial Compression Test Results

Figures 10, 11 and 12 are plots of deviator stress versus axial strain for MC-3 contents of 3, 5 and 7 per cent respectively tested at an all around pressure of 20 psi. On each of the three plots, a stress strain curve for freeze-thaw test specimens is shown, along with a curve for cured specimens and for specimens immersed for 14 days prior to testing. Only the freeze-thaw test information was obtained in this investigation, the remainder coming from Pennell's current triaxial investigations.

With specimens cured for 7 days a decrease in maximum deviator stress with increasing asphalt content occurred, similar to that found with unconfined compression tests. As with the unconfined test, the difference in maximum deviator stress for the three asphalt contents was small after specimens were subjected to soaking or freezing and thawing. For a given test, the initial tangent modulus, that is, the initial slope of the stress-strain curve, decreases with increasing asphalt content.

The dashed Mohr envelopes in Figures 13, 14 and 15, represent Pennell's results of triaxial compression tests on specimens immersed for 14 days before testing. The Mohr circles and envelopes for freeze-thaw specimens obtained in this investigation are shown on the same plot. Values of cohesion shown on these plots are very small and too nearly identical to show a trend. Bearing in mind the results of freeze-thaw tests evaluated by the unconfined test, there is

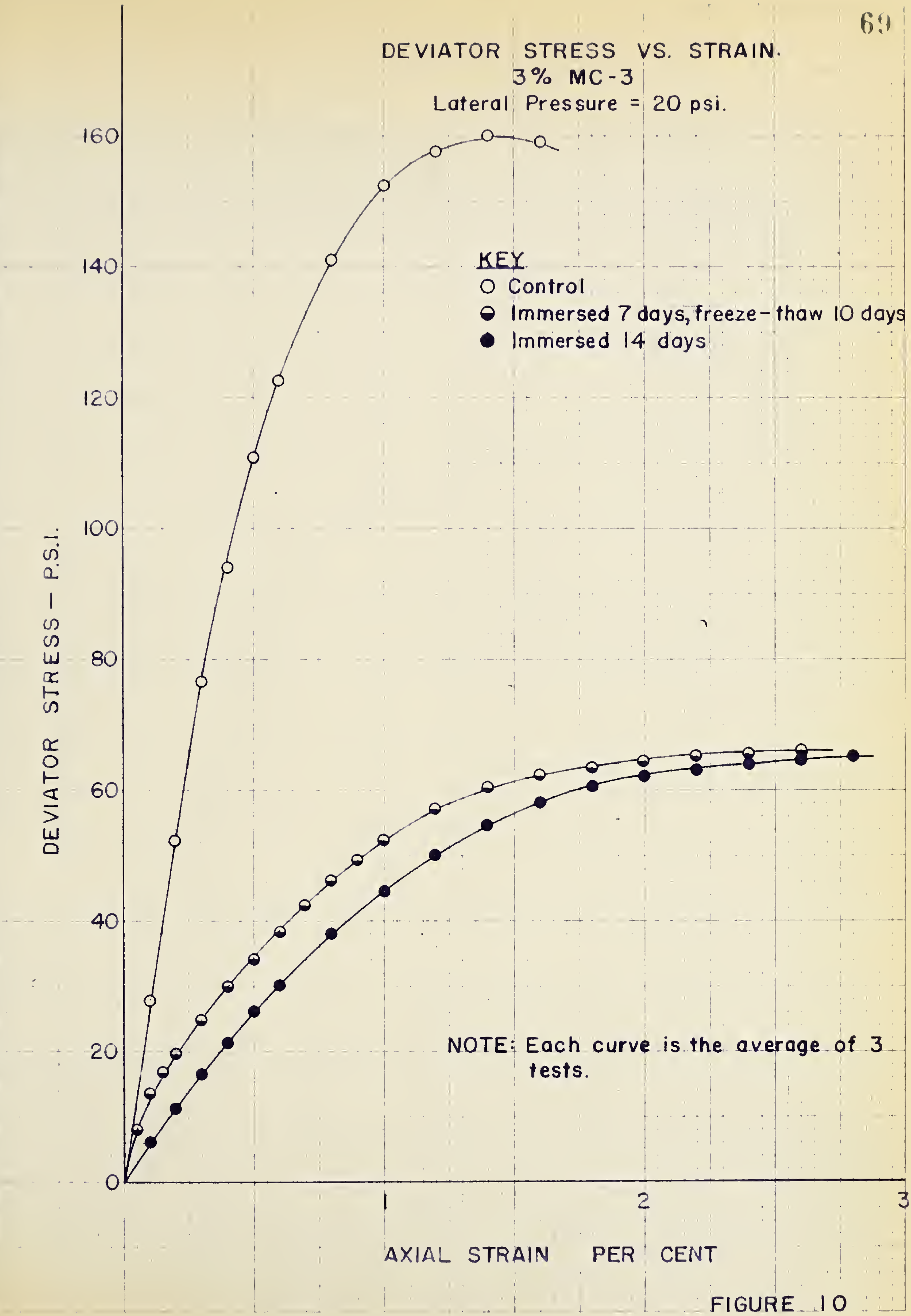

DEVIATOR STRESS VS. STRAIN.
3% MC-3
Lateral Pressure = 20 psi.
KEY
Control
Immersed 7 days, freeze-thaw 10 days
Immersed 14 days
NOTE: Each curve is the average of 3 tests.
DEVIATOR STRESS — P.S.I.
0
20
40
60
80
100
120
140
160
1
2
3
AXIAL STRAIN PER CENT

FIGURE 10

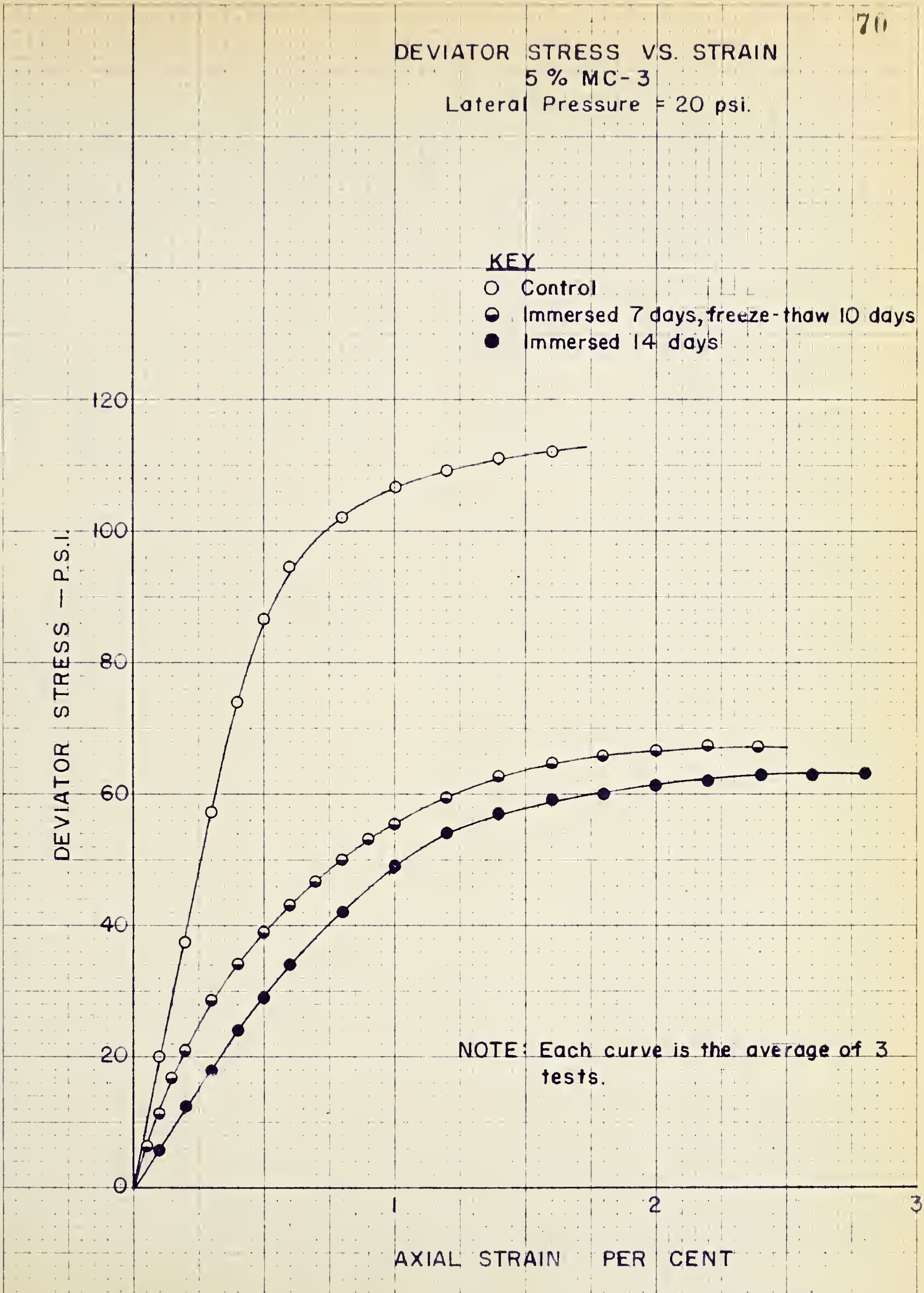

DEVIATOR STRESS VS. STRAIN
5 % MC-3
Lateral Pressure = 20 psi.
KEY
Control
Immersed 7 days, freeze-thaw 10 days
Immersed 14 days
NOTE: Each curve is the average of 3 tests.
DEVIATOR STRESS — P.S.I.
0
20
40
60
80
100
120
1
2
3
AXIAL STRAIN PER CENT

FIGURE 11

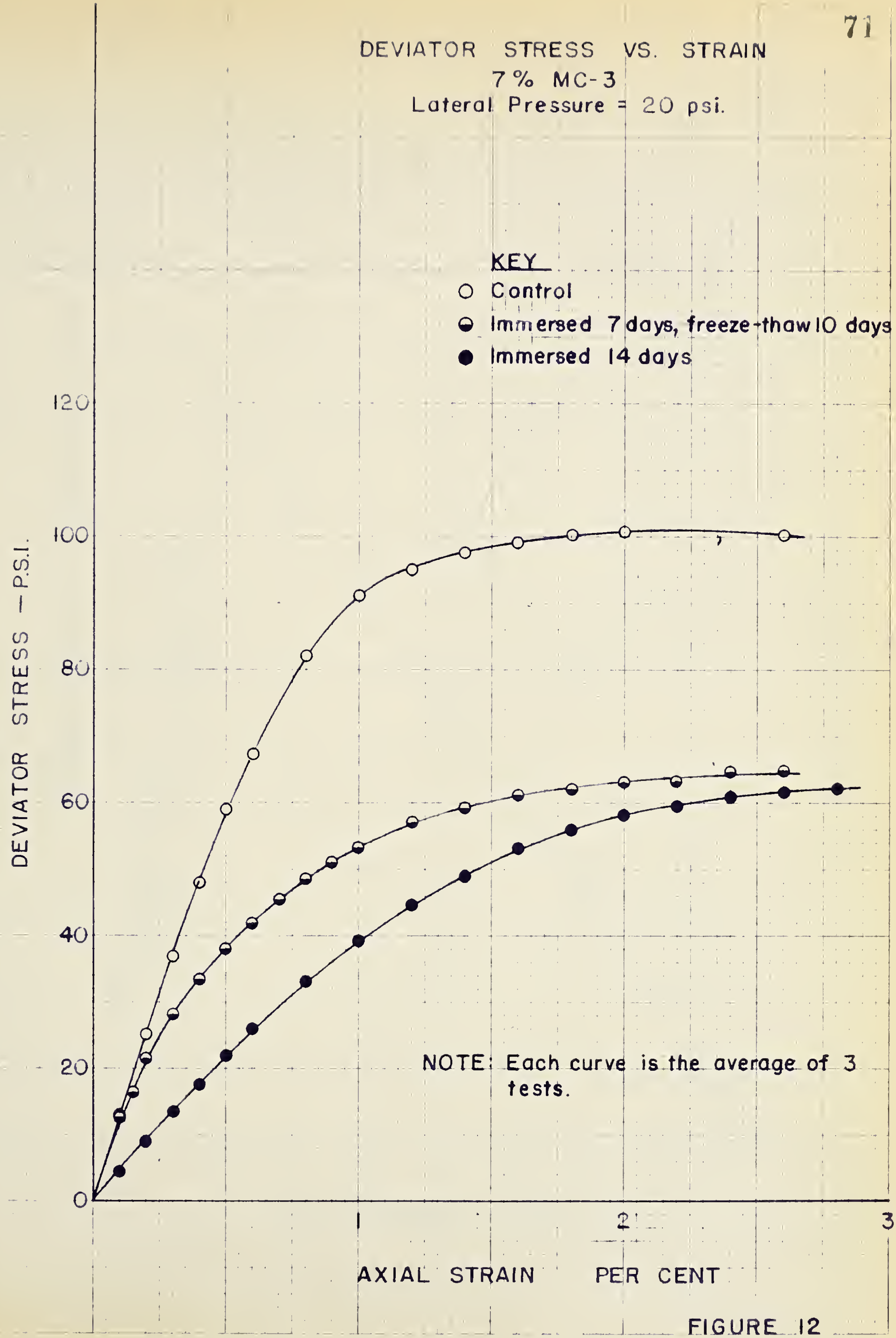

DEVIATOR STRESS VS. STRAIN
7 % MC-3
Lateral Pressure = 20 psi.
KEY
Control
Immersed 7 days, freeze-thaw 10 days
Immersed 14 days
NOTE: Each curve is the average of 3 tests.
DEVIATOR STRESS — P.S.I.
0
20
40
60
80
100
120
1
2
3
AXIAL STRAIN PER CENT

FIGURE 12

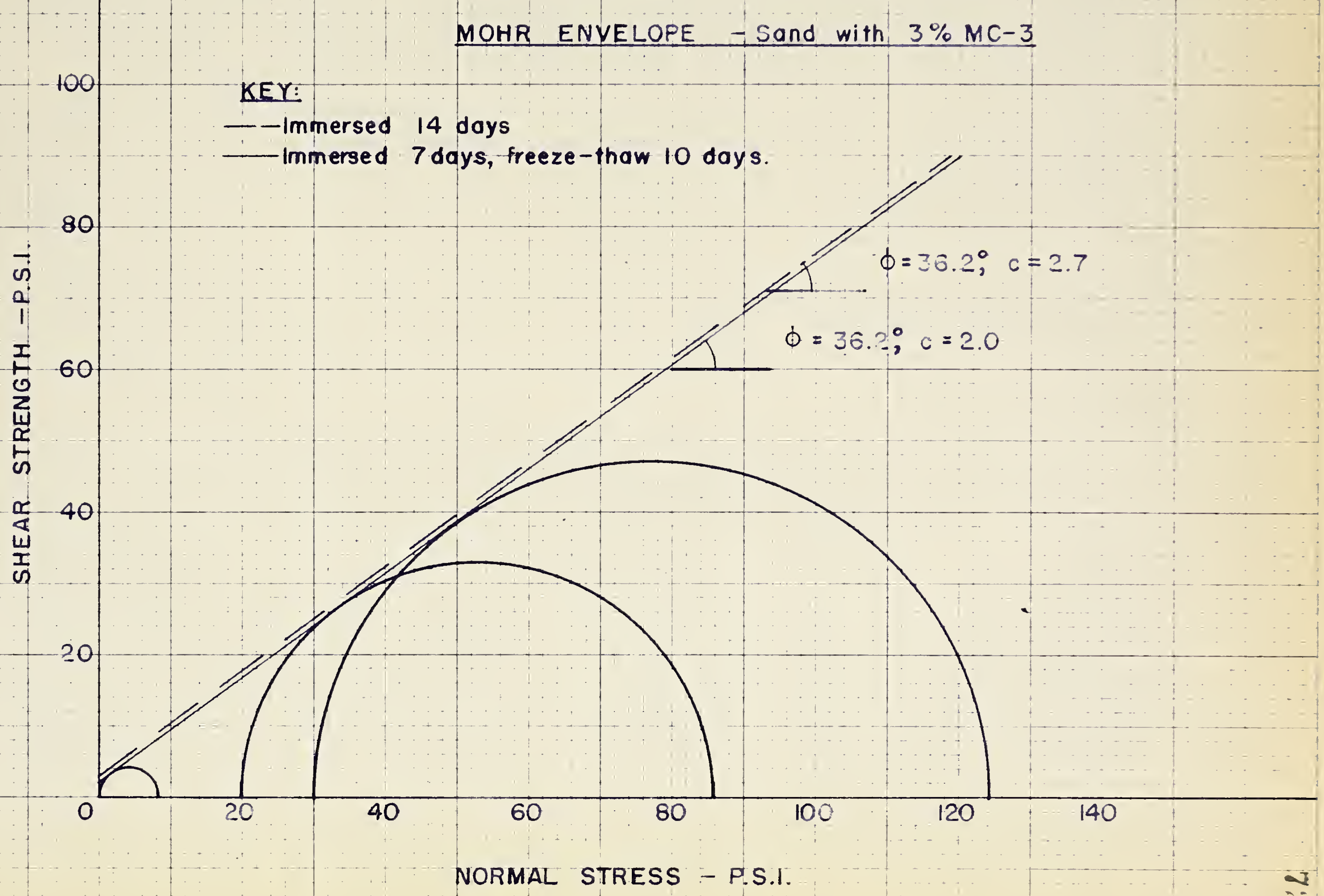

MOHR ENVELOPE - Sand with 3% MC-3
KEY:
Immersed 14 days
Immersed 7days, freeze-thaw 10 days.
ϕ = 36.2°; c = 2.7
ϕ = 36.2°; c = 2.0
SHEAR STRENGTH - P.S.I.
100
80
60
40
20
0
20
40
60
80
100
120
140
NORMAL STRESS - P.S.I.

FIGURE 13

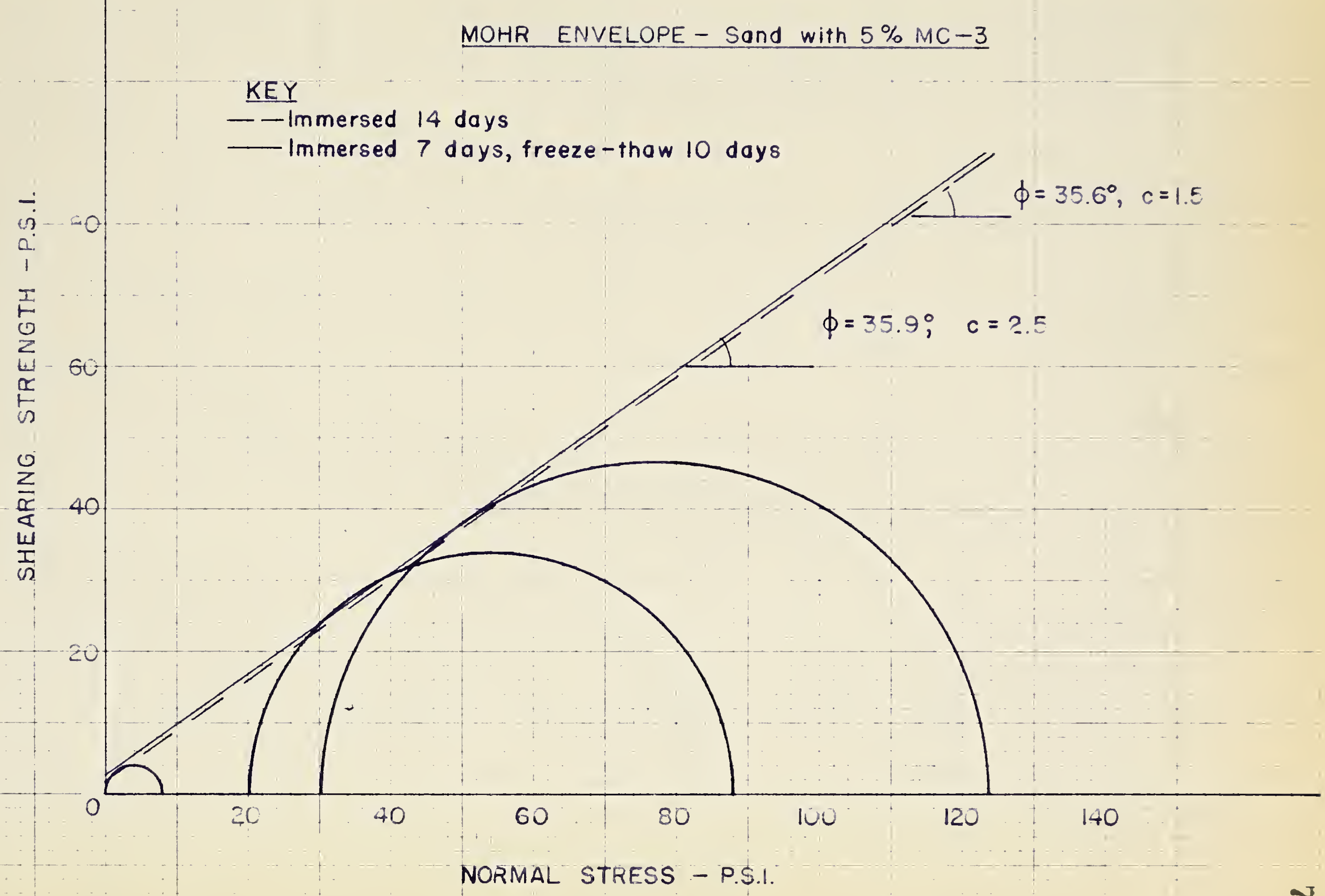
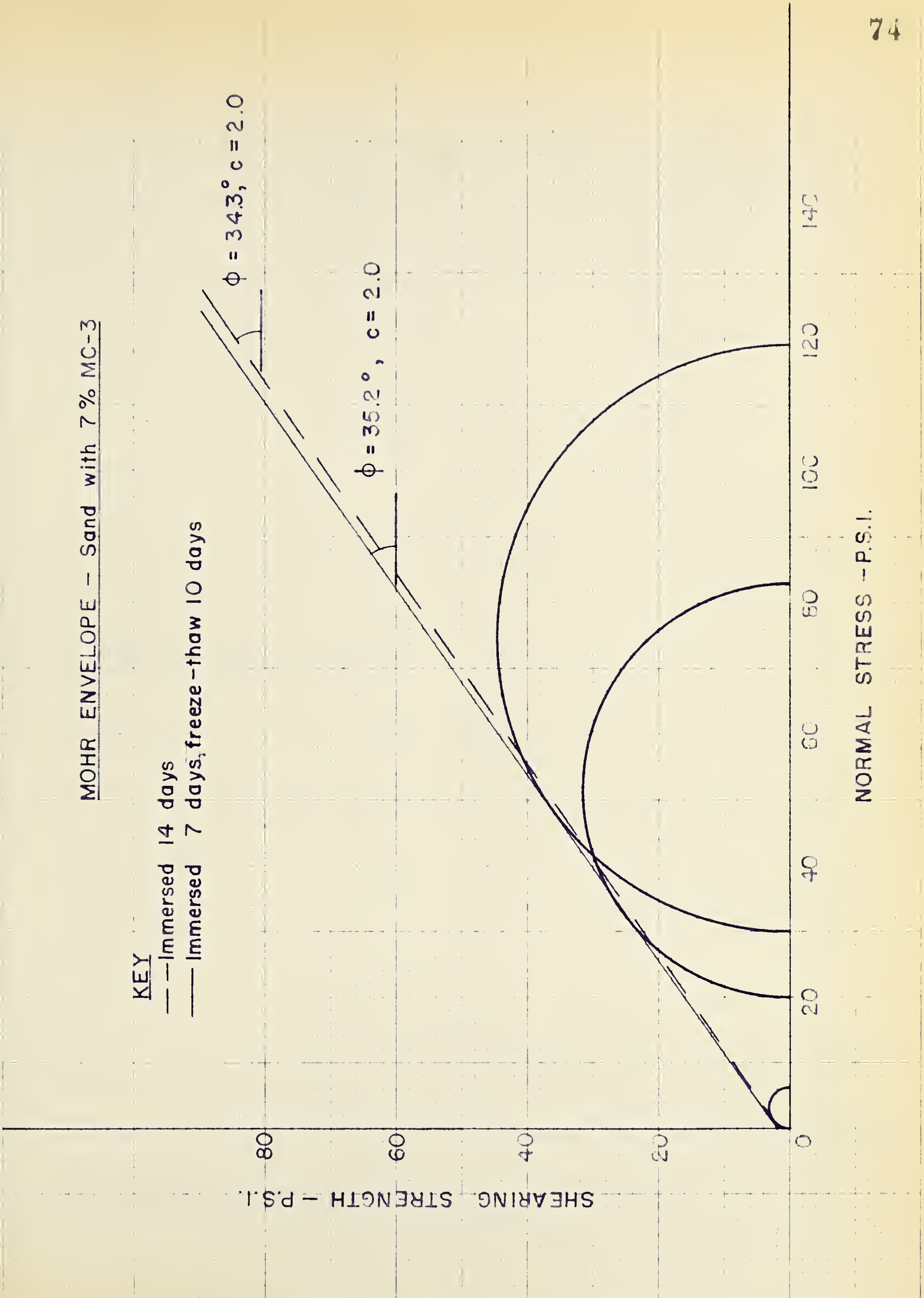

MOHR ENVELOPE - Sand with 5% MC-3
KEY
Immersed 14 days
Immersed 7 days, freeze-thaw 10 days
φ = 35.6°, c = 1.5
φ = 35.9°, c = 2.5
SHEARING STRENGTH - P.S.I.
NORMAL STRESS - P.S.I.
0
20
40
60
80
100
120
140

FIGURE 14



MOHR ENVELOPE - Sand with 7% MC-3
KEY
Immersed 14 days
Immersed 7 days, freeze-thaw 10 days
φ = 34.3°, c = 2.0
φ = 35.2°, c = 2.0
SHEARING STRENGTH - P.S.I.
80
60
40
20
0
20
40
60
80
100
120
140
NORMAL STRESS - P.S.I.

FIGURE 15

no reason to believe that the results would have been significantly different if freeze-thaw specimens had been immersed 14 days before freezing. The percentages of sand voids filled with water after 7 and 14 days were;

MC-3	7 days*	14 days**
3	31.8	50.8
5	29.3	50.8
7	29.6	42.0

A slight decrease in the angle of internal friction with increasing asphalt content, was found in this investigation for specimens soaked 7 days and subjected to 10 cycles of freezing and thawing.

Discussion of Sand - MC-3

Considering the results of the compression tests on sand-asphalt specimens, there appears to be little justification in using any more than a minimum amount of MC-3. This minimum would be the amount necessary to prevent the sand from disintegrating completely when exposed to water. Therefore the best asphalt content would be less than 5 per cent and possibly even lower than 3 per cent.

A stabilized base is normally subjected to some traffic before it is surfaced. Untreated sand will not withstand this traffic without raveling. Another function of asphalt in a stabilized base is to impart to the base the cohesion necessary

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* Average of 9 specimens each

** From Pennell; average of 12 specimens.

to withstand this wear. If a low asphalt content was selected on the basis of resistance to water immersion, the percentage of asphalt required may be greater in order to conform to this requirement.

Significance of the Durability Tests

Considering all of the conditions imposed on sand-asphalt specimens in this program, the one having the greatest effect on compressive strength was that of increasing the amount of water in the voids, after the specimens had been cured. Most of the tests employed in this study made water available to the specimens after curing by some means. These tests caused a reduction in compressive strength due to the same property, that is, an apparently poor resistance to water. The most significant test was therefore the immersion absorption test, which, for this material, was superior to the capillary absorption test.

It was noted that unconfined compression test results showed the same thing as triaxial tests in regard to a loss in strength on immersion. Therefore the unconfined compression test was seemingly as good as the triaxial test in evaluating a mix's resistance to water.

The immersion test is not significant for sand-portland cement specimens when compressive strength is used as the durability criterion. For this material, the 'open' freeze-thaw test, as evaluated by a compression test, appears to be of some value.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Since only one sand, asphalt, and cement were employed in this investigation, conclusions drawn must apply only to the methods, materials, and percentages used. Some of the conclusions may apply to sands, asphalts, and cements generally, but this was not proven.

Conclusions

1. Cured specimens decreased in compressive strength with increasing percentages of liquid asphalt.

2. The strengths of 3, 5, and 7 per cent MC-3 specimens, immersed in water for 5 and 14 days, were approximately the same and showed no strength decrease with increasing asphalt content.

3. Cured sand-asphalt specimens absorbed only a small amount of water before showing a severe loss in compressive strength. Since the effect of water was so great, the resistance to a soaking test appeared to be the most suitable durability criterion for evaluating a sand-asphalt mix.

4. Specimens with the three asphalt contents employed had nearly identical percentages of voids filled with water after 5 days, but after 14 days the water proofing function of the asphalt was apparent and less water was absorbed in specimens with higher asphalt contents.

5. Alternate heating to 150 F and cooling to 70 F caused the strength of 3 per cent MC-3 specimens to increase,

while the strengths of 5 and 7 per cent MC-3 specimens decreased.

6. Since the effect of water on the compressive strength of sand-asphalt specimens was so severe, the effect of alternate freezing and thawing alone was not determined. Variation in asphalt content did not affect the freeze-thaw resistance.

7. Sand with 8 per cent portland cement possessed a compressive strength 2 to 3 times the greatest strength obtained with sand-asphalt specimens.

8. The 'open' freeze-thaw test, similar to the British freeze-thaw test, was the only test which resulted in a lower compressive strength than the control strength, for the sand-cement specimens.

9. Triaxial compression tests on sand MC-3 specimens indicated that soaking prior to freezing and thawing causes a decrease in the angle of internal friction and the cohesion, but there was no significant effect from the freezing and thawing.

10. For a given test procedure, the initial tangent modulus of specimens tested in triaxial compression decreased with an increased asphalt content.

11. The amount of expansion on freezing of a sand-asphalt specimen was apparently dependent on the amount of water in the void spaces.

Recommendations

Rice (1949) in similar tests conducted on various Indiana sands, found that grain size, shape, and grain size distribution affected the strength of sand-asphalt considerably. With this in mind, an investigation involving different Alberta sands should be carried out in the early stages of the study of asphalt stabilization. The study involving different sands should utilize a soaking test in order that the severe effect of water on compressive strength found in this investigation can be studied further.

The strength of a granular soil is partly dependent on density. In this investigation an attempt was made to maintain a constant sand density, so the effect of density on compressive strength was not determined. A testing program should be carried out in which specimens of sand are prepared at different asphalt contents and compacted to different densities. Thus the relative effects of variation in density and variations in asphalt content on strength can be determined. The unconfined compression test appears to be suitable for this type of investigation.

The moisture contents employed in mixing and compacting specimens may be somewhat high for field use because of possible difficulties in drying the base. A laboratory investigation of the effects of various mixing and compacting moisture contents on strength and density should therefore be conducted, along with a study of laboratory curing times and temperatures, and their effects on specimen strength.

Much of the literature on soil stabilization mentions the need for correlation of laboratory tests with field conditions. In this respect, the variation of moisture content, or, in asphalt stabilization, of volatile content, with time, is of considerable importance. Therefore an investigation of field moisture content variation should be carried out as soon as asphalt stabilization is used in Alberta.

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APPENDIX A

SAND CLASSIFICATION TESTS

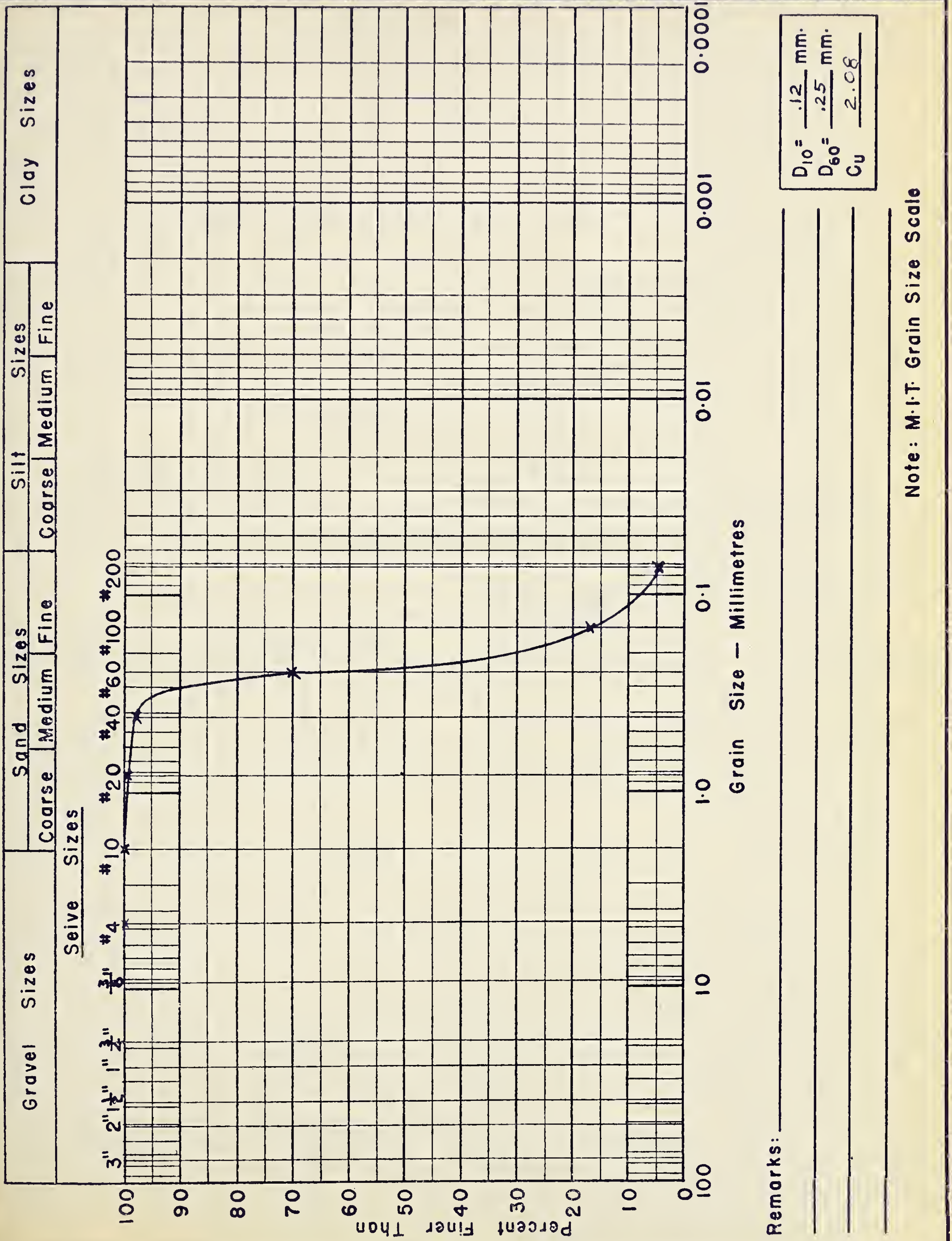
Classification tests conducted on the McGinn Pit No. 2 sand included sieve analysis and specific gravity.

The data included in Appendix A is for one sieve analysis and one specific gravity test only. The specific gravity value given in Chapter IV is the average of two tests, while the grain size distribution is the average of four determinations.

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT Thesis 81
SITE
SAMPLE Fine sand
LOCATION McGinn Pit No. 2
HOLE DEPTH
TECHNICIAN V.J. DATE Feb. 13/1962



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SOIL MECHANICS LABORATORY

SIEVE ANALYSIS

PROJECT Thesis
SITE
SAMPLE
LOCATION McGinn Pit No. 2
HOLE DEPTH
TECHNICIAN V. Jones DATE 13/2/62

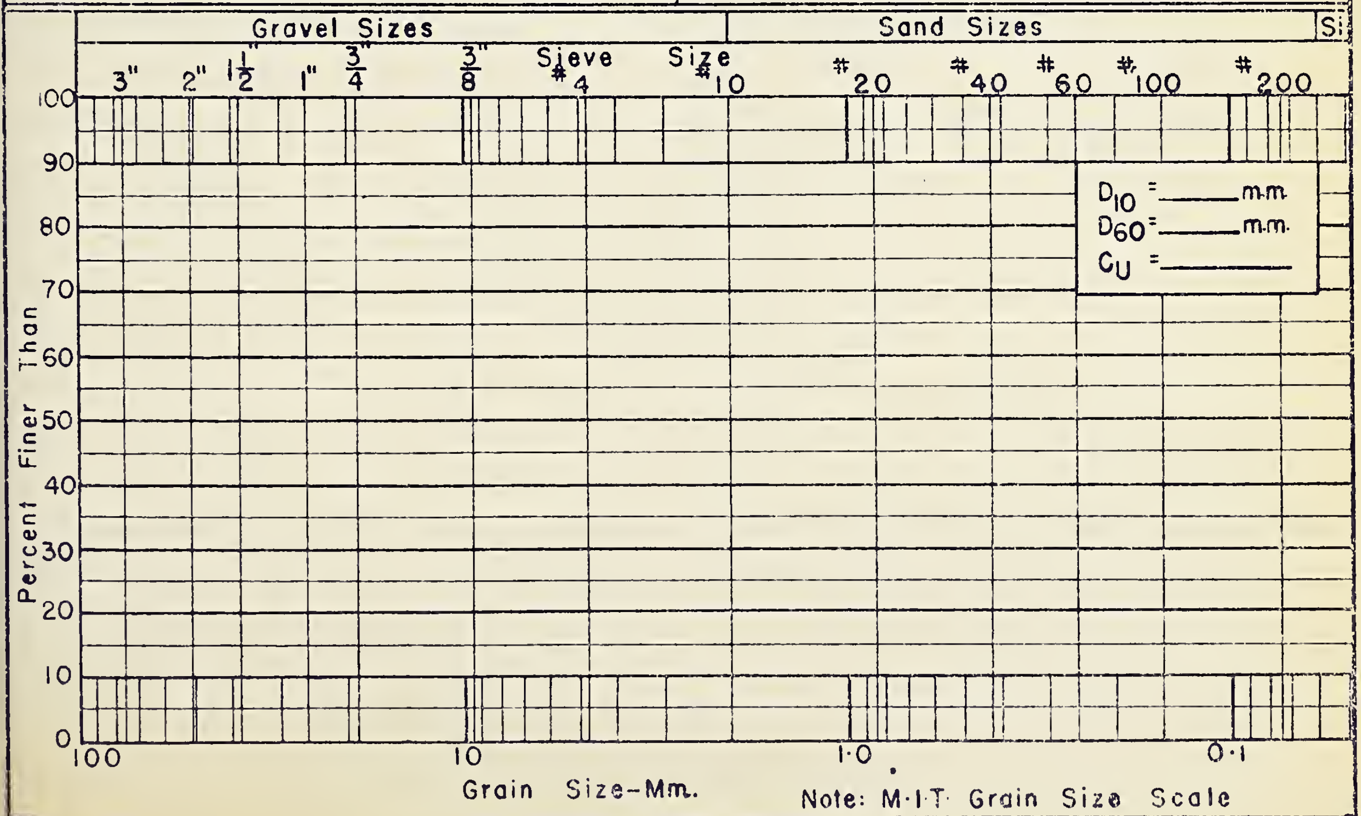
Total Dry Weight of Sample 500.0 gm	Sieve No.	Size of Opening Inches	Size of Opening Mm.	Weight Retained gms.	Total Wt. Finer Than gms.	Percent Finer Than	% Finer Than Basis Orig. Sample
Initial Dry Weight Retained No. 4							
Tare No. ______							
Wt. Dry + Tare ______							
Tare ______		3/4	19·10				
Wt. Dry ______		3/8	9·52				
	4	·185	4·76				
Passing	4						
Initial Dry Weight					500.0		
Passing No. 4	10	·079	2·000	0.1	499.9		100.0
Tare No. ______	20	·0331	·840	1.2	498.7		99.7
Wt. Dry + Tare ______	40	·0165	·420	9.4	489.3		97.9
Tare ______	60	·0097	·250	139.4	349.9		70.0
Wt. Dry ______	100	·0059	·149	264.2	85.7		17.1
	200	·0029	·074	61.1	24.6		4.9
Passing	200			24.6			

Description of Sample Brown, medium-fine sand.

Time of Sieving 10 minutes

Method of Preparation Air dry to less than 0.5% moisture

Remarks Washed through #100 and #200 sieves.



UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

SPECIFIC GRAVITY

PROJECT	Thesis
SITE	
SAMPLE	Fine Sand
LOCATION	McGinn Pit No. 2
HOLE	DEPTH
TECHNICIAN V. J.	DATE 14/2/62

Sample No.	1	
Flask No.	478	
Method of Air Removal	Vacuum	
W_{b+w+s}	784.42	
Temperature T	21.5°C	
W_{b+w}	694.03	
Evaporating Dish No.	1	
Wt. Sample Dry + Dish	952.1	
Tare Dish	807.2	
W_s	144.9	
G_s	2.66	

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles $= \dfrac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.	1		Sample No.	1	
Container No.	1		Container No.		
Wt. Sample Wet + Tare	152.64		Wt. Test Sample Wet + Tare	175.37	
Wt. Sample Dry + Tare	152.04		Tare Container	29.38	
Wt. Water	0.60		Wt. Test Sample Wet	145.99	
Tare Container	30.10		W_s (calculated)	145.2	
Wt. of Dry Soil	121.94				
Moisture Content w %	0.492%				

Description of Sample: Sand initially air dry

Remarks:

APPENDIX B

SUMMARY OF UNCONFINED COMPRESSION TEST RESULTS

1. Tabular summary of preliminary testing.
2. Tabular summary of control and durability test results for 2 inch by 2 inch, sand - MC-3 specimens.

SUMMARY OF RESULTS

Preliminary Testing

TABLE VII

Specimens cured for 120 hours at 100 F.*

Specimen	MC-3 Content %	Water Content %	Test	Average Unit Strain	Average Unit Stress psi
P1-1to3	1	14.5	Control	.267	41.2
P1-7to9	1	14.5	Freeze-Thaw	.281	29.2
P3-1to3	3	13.5	Control	.0317	41.8
P3-7to9	3	13.5	Freeze-Thaw	.0298	29.8
P5-1to3	5	12.5	Control	.0282	22.5
P5-7to9	5	12.5	Freeze-Thaw	.0277	19.9
P7-1to3	7	11.5	Control	.0326	20.3
P7-7to9	7	11.5	Freeze-Thaw	.0322	16.6
P9-1to3	9	10.5	Control	.0327	19.2
P9-7to9	9	10.5	Freeze-Thaw	.0274	15.4
P11-1to3	11	9.5	Control	.0341	15.1
P11-7to9	11	9.5	Freeze-Thaw	.0345	12.1

*Control specimens were soaked for 24 hours before the compression test. Freeze-thaw specimens were soaked for 24 hours then sealed in polyethylene before freezing and thawing.

SUMMARY OF RESULTS

Preliminary Testing

TABLE VIII

Specimens cured for 120 hours at 100 F.*

Specimen	MC-3 Content %	Water Content %	Test	Average Unit Strain	Average Unit Stress
P1-4to6	1	14.5	Control	.0414	7.4
P1-10to12	1	14.5	Freeze-Thaw	.0394	5.7
P3-4to6	3	13.5	Control	.0346	8.7
P3-10to12	3	13.5	Freeze-Thaw	.0493	6.6
P5-4to6	5	12.5	Control	.0362	11.0
P5-10to12	5	12.5	Freeze-Thaw	.0493	6.5**
P7-4to6	7	11.5	Control	.0389	9.7
P7-10to12	7	11.5	Freeze-Thaw	.0451	6.5**
P9-4to6	9	10.5	Control	.0421	11.0
P9-10to12	9	10.5	Freeze-Thaw	.0427	6.1**
P11-4to6	11	9.5	Control	.0431	9.9
P11-10to12	11	9.5	Freeze-Thaw	.0445	4.1**

*Freeze-thaw test specimens were sealed in polyethylene bags after forming, until tested in unconfined compression.

**Cracked laterally after the freeze-thaw test. Severity of cracking increasing with increasing per cent MC-3.

SUMMARY OF RESULTS

TABLE IX

Test 1 Control

Specimens 311-316, 511-516, 711-716.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1194.3	1193.3	1182.6
Volatile Content-%	14.60	12.94	10.94
Weight Sand+AC-gm.	1042.0	1056.6	1065.7
Weight AC-gm.	24.25	40.23	55.79
Weight Sand-gm.	1017.8	1016.4	1009.8
Volume Sand-cc.	382.6	382.0	379.4
Volume Sand Voids-cc.	239.6	240.2	242.8
Cured Weight-gm.	1051.2	1062.9	1072.9
Weight Water-gm. (after curing)	9.2	6.3	7.2
% Voids Water Filled (after curing)	3.84	2.62	2.96
% Voids Filled With AC	10.12	16.75	22.97
Average Compressive Strength-psi	115.2	95.5	78.2

SUMMARY OF RESULTS

TABLE X

Test 2 Capillary Absorption

Specimens 321-326, 521-526, 721-726.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1188.5	1204.2	1193.9
Volatile Content-%	13.93	13.33	11.04
Weight Sand+AC-gm.	1042.5	1062	1076
Weight AC-gm.	24.2	40.60	56.35
Weight Sand-gm.	1018.3	1021.4	1019.7
Volume Sand-cc.	382.5	383.8	383.3
Volume of Voids-cc. (sand)	239.7	238.4	238.9
Cured Weight-gm.	1050.3	1072.7	1086.9
Weight Water-gm. (after curing)	7.8	10.7	10.9
% Voids Water Filled After Curing	3.26	4.48	4.56
Soaked Weight-gm. (14 days)	1072.2	1093.8	1108.6
Weight of Absorbed Water-gm.	29.7	31.8	32.6
% Voids Water Filled After Soaking	12.40	13.33	13.65
% Voids Filled With AC	10.08	17.03	23.58
Average Compressive Strength-psi	30.2	24.7	18.3

SUMMARY OF RESULTS

TABLE XI

Test 3 Immersion Absorption

Specimens 331-336, 531-536, 731-736.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1188.0	1180.5	1181.3
Volatile Content-%	14.15	11.58	10.54
Weight Sand+AC-gm.	1040.5	1057.0	1069.0
Weight AC-gm.	24.15	39.80	55.80
Weight Sand-gm.	1016.3	1017.2	1013.2
Volume Sand-cc.	381.7	382.5	381.3
Volume Sand Voids-cc.	240.5	239.7	240.9
Cured Weight-gm.	1047.4	1063.9	1079.2
Weight Water-gm. (after curing)	6.9	6.9	10.2
% Voids Water Filled (after curing)	2.87	2.88	4.23
14 Day Soaked Weight-gm.	1184.2	1174.5	1183.3
Weight Absorbed Water-gm.	143.7	117.5	114.3
% Voids Water Filled (after soaking)	59.7	49.1	47.4
% Voids Filled With AC	10.03	16.62	23.15
Average Compressive Strength-psi	13.2	14.2	14.2

SUMMARY OF RESULTS

TABLE XII

Test 4 Heating and Cooling

Specimens 341-346, 541-546, 741-746.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1193.4	1188.0	1188.6
Volatile Content-%	14.55	12.72	11.33
Weight Sand+AC-gm.	1042.0	1053.0	1067.0
Weight AC-gm.	24.28	40.05	56.20
Weight Sand-gm.	1017.7	1013.0	1010.8
Volume Sand-cc.	382.3	381.0	380.0
Volume Sand Voids-cc.	239.9	241.2	242.2
Cured Weight-gm.	1048.4	1064.0	1079.2
Weight Water-gm. (after curing)	6.4	11.0	12.2
% Voids Water Filled (after curing)	2.67	4.56	5.03
Final Weight-gm.	1044.4	1058.6	1073.5
Final Weight Water gm.	2.4	5.6	6.5
% Voids Water Filled (after heat/cool)	1.00	2.32	2.68
% Voids Filled With AC	10.12	16.61	23.20
Average Compressive Strength-psi	130.7	76.0	50.5

SUMMARY OF RESULTS

TABLE XIII

Test 5 Closed Freeze-Thaw

Specimens 351-356, 551A-556A, 751-756.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1191.9	1199.2	1183.1
Volatile Content-%	14.58	12.98	10.77
Weight Sand+AC-gm.	1040.5	1061.0	1068.0
Weight AC-gm.	24.23	40.40	55.83
Weight Sand-gm.	1016.3	1020.6	1012.2
Volume Sand-cc.	382.0	383.5	380.7
Volume Sand Voids-cc,	240.2	238.7	241.5
Cured Weight-gm.	1049.4	1070.1	1080.5
Weight Water-gm. (after curing)	8.9	9.1	12.5
% Voids Water Filled (after curing)	3.70	3.81	5.18
5 Day Soaked Weight-gm.	1124.2	1127.7	1127.2
Weight Water-gm. (after soaking)	83.7	66.7	59.2
% Voids Water Filled (after soaking)	34.8	27.9	24.5
Final Weight-gm.	1073.8	1093.3	1090.6
Final Weight Water-gm.	33.3	32.3	22.6
Final % Voids Water Filled	13.9	13.5	9.35
% Voids Filled With AC	10.09	16.90	23.10
Average Compressive Strength-psi	27.0	40.4	24.4

SUMMARY OF RESULTS

TABLE XIV

Test 6 Open Freeze-Thaw

Specimens 361-366, 561-566, 761-766.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1192.4	1193.6	1185.2
Volatile Content-%	14.50	13.16	11.33
Weight Sand+AC-gm.	1041.0	1056.0	1064.0
Weight AC-gm.	24.23	40.25	56.0
Weight Sand-gm.	1016.8	1015.8	1008.0
Volume Sand-cc.	382.0	382.0	379.0
Volume Sand Voids-cc.	240.2	240.2	243.2
Cured Weight-gm.	1047.9	1061.5	1073.8
Weight Water-gm. (after curing)	6.9	5.5	9.8
% Voids Water Filled (after curing)	2.87	2.29	4.02
Weight After 10 Cycles F/T-gm.	1075.7	1088.2	1103.0
Weight Absorbed Water-gm.	34.7	32.2	39.0
% Voids Water Filled After F/T	14.44	13.40	16.03
% Voids Filled With AC	10.09	16.76	23.03
Average Compressive Strength-psi	37.8	40.0	37.2

SUMMARY OF RESULTS

TABLE XV

Test 7 Closed Freeze-Thaw

Specimens 371-376, 571-576, 771-776.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1192.6	1202.2	1190.1
Volatile Content-%	14.63	13.15	11.24
Weight Sand+AC-gm.	1041.5	1062.0	1070.0
Weight AC-gm.	24.23	40.50	56.2
Weight Sand-gm.	1017.3	1021.5	1013.8
Volume Sand-cc.	382.2	383.8	381.5
Volume Sand Voids-cc.	240.0	238.4	240.7
Cured Weight-gm.	1047.5	1071.9	1081.0
Weight Water-gm. (after curing)	6.0	9.9	11.0
% Voids Water Filled (after curing)	2.50	4.15	4.57
5 Day Soaked Weight (final)-gm.	1128.7	1143.3	1146.1
Final Weight Water-gm.	87.2	81.3	76.1
Final % Voids Water Filled	36.3	34.1	31.6
% Voids Filled With AC	10.10	16.98	23.33
Average Compressive Strength-psi	12.6	18.8	20.1

SUMMARY OF RESULTS

TABLE XVI

Test 8 Control

Specimens 381-386, 581-586, 781-786.

MC-3 Content-%	3	5	7
Wet Weight-gm.	1191.5	1200.2	1207.5
Volatile Content-%	15.03	13.50	11.71
Weight Sand+AC-gm.	1036.0	1057.0	1082.0
Weight AC-gm.	24.20	40.44	57.00
Weight Sand-gm.	1011.8	1016.6	1025.0
Volume Sand-cc.	380.5	382.2	385.5
Volume Sand Voids-cc.	241.7	240.0	236.7
Cured Weight-gm.	1044.9	1066.0	1091.3
Weight Water-gm. (after curing)	8.9	9.0	9.3
% Voids Water Filled (after curing)	3.69	3.75	3.93
5 Day Soaked Weight (final)-gm.	1075.9	1098.7	1123.1
Final Weight Water-gm.	39.9	41.7	41.1
Final % Voids Water Filled	16.5	17.4	17.3
% Voids Filled With AC	10.02	16.83	24.08
Average Compressive Strength-psi	16.5	17.0	15.7

APPENDIX C

Attachment of 'Demec' Points

C.I.L 'Household Cement' was employed to attach the metal discs to sand-asphalt and sand-cement specimens, The points were attached to sand-asphalt specimens immediately after 5 days of oven curing. Therefore the surface of the specimens was dry. Points were attached to sand-portland cement specimens after seven days of moist curing. The adhesive required a dry surface, therefore the centers of each end of the specimens were dried in a stream of compressed air. A thin coating of adhesive was applied to an area 1/2 inch in diameter on each end of the specimens and allowed to dry for 15 minutes. A second coat was then applied to one end of each specimen. A 'Demec' point was then visually centered on each specimen and loaded with a 5 lb. weight. After 15 minutes the specimens were inverted and points were attached to the other end in a similar manner.

APPENDIX D

SAMPLE DATA SHEETS

This section contains a sample data sheet for each of the control and durability tests conducted on the sand-asphalt mix. A sample data sheet for a triaxial compression test is also included.

UNIVERSITY OF ALBERTA

SOIL MECHANICS LABORATORY

Control Test 1 102

V. Jones Date made 22/1/62

Material Data:

Sand McGinn Pit No. 2

Asphalt 3 % MC-3

Cement -- %

Water 14.5 %

Initial Volatile Content:

Tin 2 Wet wt. - tare 120.47 gm.
Dry wt. - tare 109.03 gm.
Wt. volatiles 11.44 gm.
Tare 30.57 gm.
Wt. solids 78.46 gm.

Volatile content 14.60%

Sample No.	311	312	313	314	315	316	Total
Initial wt. gm.	199.3	199.2	198.4	199.8	198.9	198.7	1194.3
Cured wt. gm.	175.3	175.2	174.7	175.8	175.2	175.0	1051.2
Cured diam. in.	2.010	2.016	2.015	2.017	2.014	2.015	
Cured ht. in.	2.013	2.014	2.004	2.014	2.010	2.010	
Soaked wt. gm.							
Soaked diam. in.							
Soaked ht. in.							
Area sq. in.	3.19	3.19	3.19	3.20	3.18	3.19	
Strain dial	.0700	.0650	.0630	.0730	.0730	.0670	
Stress dial	.0289	.0265	.0261	.0284	.0272	.0274	
Unit strain	.0206	.0192	.0185	.0223	.0229	.0198	
Corr. Area	3.26	3.25	3.26	3.27	3.25	3.25	
Load lb.	396	362	356	389	372	375	Average
Stress psi.	121.5	111.4	109.2	119.0	114.5	115.4	115.2

Remarks: Cured 5 days at 100 F. Tested Feb. 1/62

UNIVERSITY OF ALBERTA	Capillary...Absorption Test
SOIL MECHANICS LABORATORY	V. Jones Date made. 12/1/62.

Material Data:

Sand McGinn. Pit. No.. 2.

Asphalt ....3...MC-3

Cement ......%

Water ...14.5..%

Initial Volatile Content:

Tin. 5.: Wet wt.+ tare. 84.41 gm.
Dry wt.+ tare. 76.90 gm.
Wt. volatiles. 7.51 gm.
Tare 22.98 gm.
Wt. solids 53.92 gm.

Volatile content 13.93 %

Sample No.	321	322	323	324	325	326	Total
Initial wt. gm.	198.3	198.9	197.8	198.1	197.7	197.7	1188.5
Cured wt. gm.	175.0	175.7	174.8	175.1	174.8	174.9	1050.3
Cured diam. in.	2.012	2.012	2.009	2.012	2.016	2.013	
Cured ht. in.	2.015	2.016	2.013	2.015	2.012	2.023	Total
Soaked wt 1day	177.2	177.9	177.0	177.2	177.0	177.0	
2	177.5	178.1	177.2	177.6	177.2	177.2	
3	177.8	178.4	177.4	177.8	177.5	177.5	
4	178.1	178.6	177.7	178.0	177.6	177.8	
5	178.1	178.7	177.7	178.0	177.6	177.9	
6	178.4	179.0	178.0	178.3	177.9	178.2	
7	178.4	179.1	178.0	178.4	177.9	178.2	
8	178.5	179.2	178.1	178.4	177.9	178.2	
9	178.4	179.2	178.2	178.4	177.9	178.2	
10							
11							
12							
13	178.8	179.6	178.4	178.7	178.2	178.6	
Final wt. 14	178.8	179.6	178.3	178.8	178.2	178.5	1072.2
Height in	2.008	2.019	2.014	2.019	2.015	2.016	
Diameter in							
Area sq. in	3.18	3.18	3.17	3.18	3.19	3.18	
Strain dial	.0530	.0500	.0450	.0510	.0520	.0490	
Stress dial	.0072	.0071	.0071	.0073	.0072	.0071	
Unit Strain	.0229	.0215	.0190	.0219	.0224	.0209	
Corr. Area	3.25	3.25	3.23	3.25	3.26	3.25	
Load lb.	98.5	97.5	97.5	100.0	98.5	97.5	Average
Stress psi.	30.3	30.0	30.2	30.8	30.2	30.0	30.2

Remarks: Wet for approx. 1/4 inch (inside & out)
Intersection of failure cones seems to be approx. at center of dry portion.

UNIVERSITY OF ALBERTA

SOIL MECHANICS LABORATORY

Immersion Absorption Test 3 164

V. Jones Date made 9/1/62..

Material Data:

Sand McGinn Pit No. 2.

Asphalt ...3.... MC-3

Cement ..**....

Water ..14.5..

Initial Volatile Content:

Tin 1.: Wet wt. + tare	100.26 gm.
Dry wt. + tare	90.17 gm.
Wt. volatiles	10.09 gm.
Tare	18.85 gm.
Wt. solids	71.32 gm.

Volatile content 14.15.

Sample No.		331	332	333	334	335	336	Total
Initial wt.	gm.	198.8	198.3	198.3	197.5	198.0	197.1	1188.0
Cured wt.	gm.	175.1	174.8	174.8	174.0	174.7	174.0	1047.4
Cured diam.	in.	2.016	2.02	2.018	2.023	2.020	2.020	
Cured ht.	in.	2.02	2.018	2.023	2.015	2.020	2.024	Total
Soaked wt 1 day		178.8	178.7	178.3	177.6	178.4	177.7	
2		180.5	180.5	180.2	179.5	180.4	179.8	
3		181.0	181.0	180.8	180.1	181.1	180.1	
4		182.7	182.9	182.8	181.8	182.9	182.0	
5		183.4	183.7	183.4	182.7	183.7	182.7	
6		186.0	186.7	186.3	185.7	186.7	185.2	
7		188.0	188.7	188.4	187.8	188.7	187.2	
8		189.7	190.6	190.6	190.0	190.6	188.9	
9		191.1	191.8	192.2	191.3	192.0	190.6	
10		192.9	193.6	193.8	192.7	193.4	192.2	
11		194.9	195.2	195.3	194.5	195.1	194.0	
12		195.6	195.7	196.0	195.3	196.0	194.8	
13								
Final wt. 14		197.9	197.4	197.7	196.8	197.5	196.9	1184.2
Height	in	2.020	2.02	2.02	2.01	2.02	2.02	
Diameter	in							
Area	sq. in	3.19	3.20	3.20	3.22	3.20	3.20	
Strain dial		.0800	.0800	.070	.070	.066	.065	
Stress dial		.0029	.0033	.0032	.0034	.0031	.0028	
Unit Strain		.0385	.0383	.0334	.0333	.0315	.0311	
Corr. Area		3.31	3.33	3.31	3.33	3.31	3.31	
Load	lb.	41.0	46.5	45.0	47.5	43.5	40.0	Average
Stress	psi.	12.4	13.96	13.6	14.26	13.14	12.08	13.2

Remarks: Cured 5 days, Soaked 14 days. Water changed after day 11.

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SOIL MECHANICS LABORATORY

Heating and Cooling Test

V. Jones Date made 26/1/62

Material Data:

Sand McGinn Pit No. 2.

Asphalt ....3....% MC-3

Cement ....--....%

Water 14.5.%

Initial Volatile Content:

Tin 3 : Wet wt. + tare 107.12 gm.
Dry wt. + tare 97.48 gm.
Wt. volatiles 9.64 gm.
Tare 31.28 gm.
Wt. solids 66.20 gm.

Volatile content 14.55%

Sample No.		341	342	343	344	345	346	Total
Initial wt. gm.		199.9	198.8	199.2	198.6	198.4	198.5	1193.4
Cured wt. gm.		175.3	174.6	175.0	174.6	174.4	174.5	1048.4
Cured diam. in.		2.015	2.013	2.013	2.013	2.015	2.015	
Cured ht. in.		2.010	2.008	2.014	2.006	2.008	2.005	
Ref. length in.		2.131	2.127	2.137	2.126	2.130	2.127	
Wt. + points gm.								
xxxxxxxxxxxxxx								
xxxxxxxxxxxxxx								
Ref. length in.		2.131	2.127	2.137	2.126	2.130	2.127	Avg.chg.
Cycle 1	H	2.132	2.127	2.137	2.126	2.130	2.127	+.0002
	C	2.131	2.127	2.137	2.125	2.130	2.126	-.0003
2	H	2.133	2.128	2.137	2.127	2.130	2.126	+.0005
	C	2.131	2.129	2.136	2.127	2.129	2.125	-.0002
3	H	2.133	2.127	2.137	2.126	2.130	2.126	+.0002
	C	2.132	2.129	2.136	2.127	2.129	2.126	+.0002
4	H	2.132	2.127	2.137	2.125	2.129	2.126	-.0003
	C	2.1325	2.129	2.137	2.127	2.130	2.127	+.0010
5	H	2.133	2.127	2.137	2.1275	2.130	2.127	+.0006
	C	2.1315	2.127	2.136	2.126	2.129	2.1255	-.0005
6	H	2.1315	2.128	2.1375	2.1265	2.130	2.126	+.0002
	C	2.132	2.1285	2.1365	2.1265	2.1295	2.126	+.0002
7	H	2.1335	2.129	2.1375	2.1265	2.130	2.126	+.0008
	C	2.132	2.129	2.136	2.126	2.1295	2.126	+.0001
8	H	2.132	2.127	2.1365	2.1265	2.1295	2.126	+.0001
	C	2.1325	2.128	2.1365	2.1255	2.1295	2.126	.0000
9	H	2.132	2.127	2.1365	2.1255	2.129	2.1255	-.0006
	C	2.1325	2.1285	2.1355	2.1245	2.1295	2.1255	-.0003
10	H	2.133	2.128	2.137	2.126	2.130	2.126	+.0003
	C	2.132	2.1265	2.1365	2.125	2.1285	2.125	-.0008
Final wt+points								Total
Final wt. gm.		174.8	173.9	174.3	173.8	173.8	173.8	1044.4
Length in.		2.011	2.008	2.013	2.005	2.006	2.003	
Diam. in.								
Area sq.in.		3.19	3.18	3.18	3.18	3.19	3.19	
Strain dial		.0740	.0700	.0700	.0670	.0740	.0740	
Stress dial		.0316	.0311	.0311	.0320	.0308	.0309	
Unit Strain		.0212	.0195	.0195	.0175	.0216	.0216	
Corr. Area		3.26	3.25	3.25	3.24	3.26	3.26	
Load lb.		430	423	423	435	420	421	Average
Stress psi.		131.9	130.1	130.1	134.3	128.8	129.2	130.7

Remarks:

UNIVERSITY OF ALBERTA	Freeze - Thaw Test 5
SOIL MECHANICS LABORATORY	V. Jones Date made 27/1/62

Material Data:

Sand. McGinn Pit No. 2.

Asphalt ...3.....% MC-3

Cement ...--....%

Water ...14.5..%

Initial Volatile Content:

Tin. 1.: Wet wt.+ tare..106.22.gm.
Dry wt.+ tare...95.10.gm.
Wt. volatiles..11.12.gm.
Tare ..18.85.gm.
Wt. solids ..76.25.gm.

Volatiles content 14.58

Sample No.	351	352	353	354	355	356	Total
Initial wt. gm.	199.1	198.3	199.3	199.3	198.3	197.6	1191.9
Cured wt. gm.	175.2	174.5	175.3	175.6	174.8	174.0	1049.4
Cured diam in.	2.016	2.022	2.021	2.021	2.027	2.020	
Cured ht. in.	2.020	2.010	2.019	2.020	2.011	2.022	
Ref. length in.	2.126	2.122	2.136	2.137	2.125	2.132	
Spec. wt+points	176.2	175.4	176.3	176.6	175.7	175.0	
Wet wt.+points	188.5	188.0	188.8	189.2	188.2	187.3	
Soaked wt. gm.	187.5	187.0	187.8	188.2	187.3	186.3	1124.2
Ref. length in.	2.131	2.1265	2.1425	2.144	2.129	2.1375	Avg. chg.
Cycle 1 F	2.139	2.1295	2.1455	2.1495	2.136	2.1405	+.0049
T	2.132	2.1275	2.1435	2.1445	2.130	2.1385	+.0009
2 F	2.142	2.1335	2.149	2.1495	2.136	2.142	+.0069
T	2.130	2.123	2.1425	2.143	2.129	2.1375	-.0001
3 F	2.138	2.1335	2.149	2.152	2.138	2.142	+.0070
T	2.130	2.128	2.142	2.144	2.1295	2.138	+.0002
4 F	2.138	2.133	2.1455	2.1515	2.1375	2.141	+.0060
T	2.130	2.127	2.142	2.144	2.129	2.137	-.0002
5 F	2.1395	2.131	2.142	2.151	2.1355	2.1385	+.0045
T	2.1315	2.127	2.1415	2.143	2.129	2.1355	-.0005
6 F	2.135	2.130	2.143	2.147	2.1315	2.1355	+.0019
T	2.130	2.1255	2.1395	2.142	2.127	2.134	-.0021
7 F	2.132	2.1235	2.1385	2.147	2.132	2.1355	+.0005
T	2.1295	2.1255	2.139	2.142	2.1275	2.134	-.0022
8 F	2.131	2.123	2.138	2.1465	2.129	2.1325	-.0009
T	2.128	2.1255	2.1375	2.1415	2.127	2.134	-.0028
9 F	2.126	2.126	2.138	2.1465	2.1255	2.132	-.0028
T	2.127	2.1245	2.139	2.141	2.1265	2.1335	-.0032
10 F	2.126	2.123	2.137	2.143	2.125	2.131	-.0042
T	2.1265	2.1235	2.139	2.1405	2.126	2.133	-.0037
Final wt+points							Total
Final wt. gm.	179.5	178.9	179.2	180.6	178.3	177.3	1073.8
Length in.	2.020	2.011	2.022	2.017	2.012	2.016	
Diam. in.							
Area sq.in.	3.19	3.21	3.21	3.21	3.23	3.205	
Strain dial	.0510	.0620	.0610	.0590	.0560	.0560	
Stress dial	.0060	.0062	.0067	.0058	.0068	.0073	
Unit strain	.0225	.0279	.0271	.0266	.0246	.0243	
Corr. Area	3.26	3.30	3.30	3.30	3.31	3.29	
Load lb.	83	85.5	92	80.5	93.5	100	Average
Stress psi.	25.5	25.9	27.9	24.4	28.2	30.4	27.0

Remarks: Measuring points added after curing, then soak 5 days and freeze-thaw 10 days.

[illegible] Freeze - Thaw Test 6

[illegible] V. Jon[illegible] Date [illegible] 3/2/62

Material Data:

Source .McGinn Pit No..2.

Asphalt ...3....%

Cement ....--....

Water ....14.5....

Initial Volatile Content:

Sn. 3: Wet wt.+ tare ..141.12 gm.

Dry wt.+ tare ..127.21

Wt. volatiles ..13.91

Tare ..31.28

Wt. solids ..95.53

Volatiles content % 14.56

Sample No.		361	362	363	364	365	366	Total
Initial wt.	gm.	198.6	198.8	199.0	199.1	198.9	198.0	1192.4
Cured wt.	gm.	174.4	174.6	174.9	175.0	174.9	174.1	1047.9
Cured dia.	in.	2.017	2.015	2.015	2.017	2.017	2.017	
Cured ht.	in.	2.008	2.014	2.016	2.015	2.015	2.003	
Ref. length	in.							
[illegible] wt.+ pins								
Wet wt.+ pins								
Cleaned wt.	gm.							
Ref. length	in.							Avg. [illegible]
Cycle 1	F							
	T							
2	F							
	T							
3	F							
	T							
4	F							
	T							
5	F							
	T							
6	F							
	T							
7	F							
	T							
8	F							
	T							
9	F							
	T							
10	F							
	T							
Final wt+points								Total
Final wt.	gm.	180.7	180.4	178.5	179.5	178.6	178.0	1075.2
Length	in.	2.017	2.024	2.028	2.027	2.020	2.016	
Diam.	in.							
Area	sq. in.	3.19	3.19	3.19	3.19	3.19	3.19	
Strain	in.	.0610	.0650	.0560	.0590	.0600	.0530	
Stress dial		.0082	.0086	.0091	.0093	.0097	.0091	
Unit strain		.0264	.0282	.0235	.0248	.0252	.0219	
Corr. area		3.27	3.28	3.27	3.27	3.27	3.26	
Load	lb.	113	118.5	125	128	133	125	Average
Stress	psi.	34.6	36.1	33.2	39.2	40.6	38.4	37.8

Remarks: Modified British Standard 1924 test. Cured 5 days, freeze-thaw in open system for 10 days. After F/T the bottom end of 3% specimens showed more deterioration than 5 and 7 % specimens.

UNIVERSITY OF ALBERTA | Freeze - Thaw Test 7
SOIL MECHANICS LABORATORY | V. Jor[illegible] Date made 12/2/62

Material Data:

Sand McGinn Pit No. 2

Asphalt ...3...% RC-3

Cement ...--...

Water ...14.5.

Initial Volatile Content:

Tin.1.: Wet wt.+ tare..93.62.gm.
Dry wt.+ tare..84.08.
Wt. volatiles...9.54...
Tare ..18.85..
Wt. solids ..65.23..

Volatiles content 14.63%

Sample No.		371	372	373	374	375	376	Total
Initial wt.	gm.	197.7	198.0	199.4	199.0	199.0	199.5	1192.6
Cured wt.	gm.	173.5	173.9	175.1	174.8	174.9	175.3	1047.5
Soaked diam.	in.	2.023	2.023	2.021	2.023	2.019	2.023	
Soaked ht.	in.	2.022	2.025	2.023	2.022	2.026	2.026	
Ref. length	in.							
Spec. wt+points								
Net wt.+points								
Soaked wt.	gm.	186.4	187.3	188.9	187.8	188.7	189.6	1128.7
Ref. length	in.							Av. [illegible]
Cycle 1	F							
	T							
2	F							
	T							
3	F							
	T							
4	F							
	T							
5	F							
	T							
6	F							
	T							
7	F							
	T							
8	F							
	T							
9	F							
	T							
10	F							
	T							
Final wt+points								Total
Final wt.	gm.	186.5	187.1	189.0	187.7	189.3	189.7	1128.7
Length	in.	2.012	2.014	2.030	2.028	2.026	2.026	
Diam.	in.	2.018	2.020	2.016	2.019	2.022	2.020	
Area	sq. in.	3.22	3.205	3.19	3.18	3.205	3.205	
Strain dial		.0900	.1000	.0850	.0900	.0930	.0820	
Stress dial		.0032	.0031	.0029	.0029	.0028	.0030	
Unit strain		.0434	.0484	.0410	.0435	.0451	.0395	
Corr. area		3.36	3.36	3.33	3.32	3.36	3.33	
Load	lb.	44.8	43.5	41.0	41.0	40.0	42.5	Average
Stress	psi.	13.3	12.9	12.3	12.3	11.9	12.8	12.6

Remarks: Soaked 5 days, then sealed in polyethylene bags for 10 cycles of freezing and thawing.

UNIVERSITY OF ALBERTA

SOIL MECHANICS LABORATORY

Control Test 8

V. Jones

Date made 13/3/62

Material Data:

Sand McGinn Pit No. 2

Asphalt 3 % MC-3

Cement -- %

Water 14.5 %

Initial Volatile Content:

Tin 1 Wet wt. - tare 100.83 gm.
Dry wt. - tare 90.12 gm.
Wt. volatiles 10.71 gm.
Tare 18.85 gm.
Wt. solids 71.27 gm.

Volatile content 15.03 %

Sample No.	381	382	383	384	385	386	Total
Initial wt. gm.	196.7	198.5	199.1	198.4	199.4	199.4	1191.5
Cured wt. gm.	172.3	174.0	174.6	174.1	174.9	175.0	1044.9
Cured diam. in.							
Cured ht. in.							
Soaked wt. gm.	177.3	179.3	179.7	179.4	180.1	180.1	1075.9
Soaked diam. in.	2.026	2.026	2.029	2.024	2.021	2.010	
Soaked ht. in.	1.975	1.995	1.990	1.994	2.006	2.012	
Area sq. in.	3.23	3.23	3.24	3.22	3.21	3.17	
Strain dial	.063	.060	.070	.062	.059	.063	
Stress dial	.0040	.0038	.0036	.0039	.0042	.0043	
Unit strain	.0295	.0281	.0032	.0290	.0274	.0293	
Corr. Area	3.33	3.32	3.35	3.32	3.30	3.27	
Load lb.	55.0	52.5	50.0	53.5	57.5	59.0	Average
Stress psi.	16.5	15.8	14.9	16.6	17.4	18.0	16.5

Remarks: Cured 5 days, Soaked 5 days.

UNIVERSITY OF ALBERTA
DEPT. OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SAND-ASPHALT STABILIZATION
TRI-AXIAL & COMPACTION DATA
3% MC-3, 14.5% Water

Project Thesis
Site
Sample McGinn Pit No. 2
Technician V. Jones
Date specimens made: 14/2/6[illegible]
Cure 7 days, Soak 7 days, T/I 1[illegible]

Specimen Number	31	32			
Asphalt Content %	3	3			
Lateral Pressure--psi - σ_{III}	20	20			
Length--in.	6.446	6.439			
Area--in²	6.20	6.20			
Volume--ft³	.0231	.0231			
Wet Wt. After Compaction	1246.1	1243.0			
Wet Unit Weight					
Dry Unit Weight					
Gs = 2.65 Volume Soil Solids					
Void Ratio					
Curing Time--hrs.	168	168			
Wt. Sample Before Testing (wet)	1185.3	1182.9			
Dry Weight of Sample					
% of Vol. Filled After Comp.					
Vol. Water Content at Comp.					
% of Vol. Filled at Testing					
Vol. Water Content at Testing					
Strain Rate in./min.	.01	.01			
Cured Weight -gm.	1096.8	1097.1			

Prov. Ring Dial	Load lb.	Strain Read.	Strain (Unit)	Corr. Area in²	σ_I psi	$\sigma_I-\sigma_{III}$ psi	Prov. Ring Dial	Load	Strain Read.	Strain (Unit)	Corr. Area	σ_I	$\sigma_I-\sigma_{III}$
25	10.3	0032	0005	6.15		3.9	0						
63	27.1	0065	0010	6.15		9.7	46	19.8	32.5	0005	6.15		7.1
84	36.2	0097	0015	6.16		13.0	71	30.6	65	001	6.15		11.[illegible]
103	44.4	0130	0020	6.16		15.9	91	39.2	97.5	0015	6.16		14.[illegible]
122	52.7	0162	0025	6.17		18.8	109	47.0	130	002	6.16		16.8
137	59.4	0195	0030	6.17		21.2	141	61.1	195	003	6.17		21.[illegible]
153	66.4	0227	0035	6.17		23.7	173	75.1	260	004	6.18		26.[illegible]
171	74.2	0260	0040	6.18		26.5	202	87.3	325	005	6.18		31.[illegible]
186	80.9	0292	0045	6.18		28.9	229	99.8	390	006	6.19		35.6
201	87.4	0325	0050	6.18		31.2	256	111.7	455	007	6.19		38.[illegible]
214	93.0	0357	0055	6.19		33.1	281	122.4	520	008	6.20		43.[illegible]
223	99.4	0390	0060	6.19		35.4	304	133.0	585	009	6.21		47.2
254	110.7	0455	0070	6.19		39.4	324	141.7	650	010	6.21		50.3
279	121.6	0520	0080	6.20		43.2	358	156.6	780	012	6.22		55.5
301	131.6	0585	0090	6.21		46.7	382	167	910	014	6.24		5[illegible]
321	140.4	0650	0100	6.21		49.9	395	172.7	1040	016	6.25		61.[illegible]
353	154.4	0780	0120	6.22		54.8	404	176.8	1170	018	6.26		62.3
378	165.3	0910	0140	6.24		55.4	411	179.7	1300	020	6.27		63.2
396	173.3	1040	0160	6.25		61.1	416	182	1430	022	6.28		63.[illegible]
403	175.7	1170	0180	6.26		62.9	418	183.1	1560	024	6.30		64.[illegible]
413	180.7	1300	0200	6.27		63.6	423	185.2	1690	026	6.32	34.6	64.6
419	183.3	.1430	.022	6.28	34.1	64.9							

[illegible]

APPENDIX E

COMPACTIVE EFFORT VS. DRY UNIT WEIGHT

In the preliminary stages of this investigation it was necessary to establish the compactive effort necessary to give the sand a dry unit weight of 104 lbs. per cubic foot when compacted at 15% moisture. That is, to achieve approximately the same densities as Pennell obtained with his 2.8 inch by 6.5 inch specimens. Figure 16 illustrates the results of the trial and error procedure. Specimens were compacted to about 2 ½ inches height in a 2 inch by 6 inch mold, using a 5 lb. hammer dropping 12 inches. The compactive effort selected for the testing program was 49,500 ft. lbs. 1 cu. ft. or 36 blows of the hammer. The above compactive effort is based on a 2 inch by 2 inch specimen.

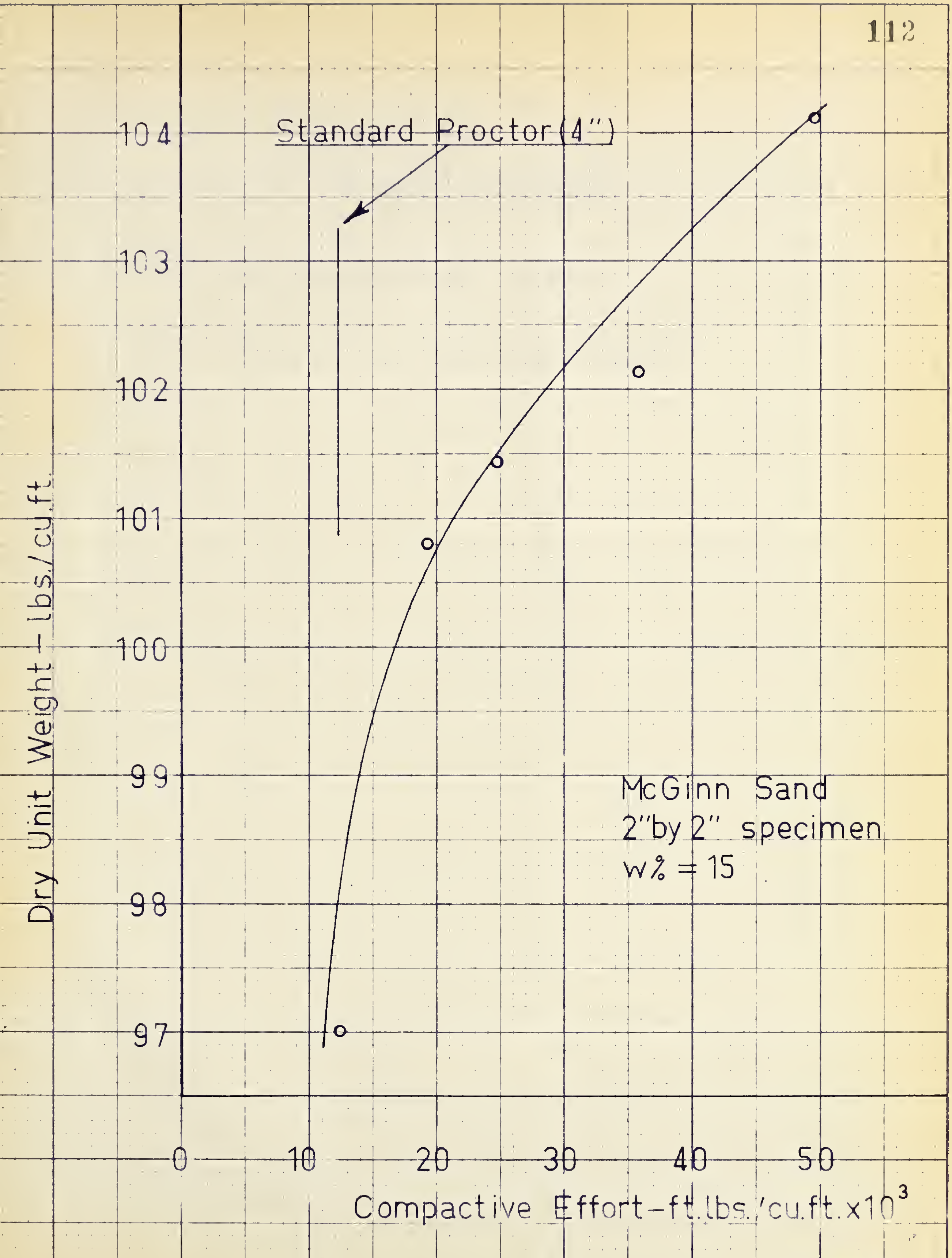
Standard Proctor (4")
104
103
102
101
100
99
98
97
Dry Unit Weight - lbs./cu.ft.
McGinn Sand
2" by 2" specimen
w% = 15
0
10
20
30
40
50
Compactive Effort - ft.lbs./cu.ft. x10^3

Figure 16

APPENDIX F

SPECIMEN PREPARATION

1. 2 inch Sand-Asphalt Specimens
2. 2 inch Sand-Portland Cement Specimens
3. Triaxial Compression Test Specimens

1. 2 inch Sand-MC-3 Specimens

1. The weight of material necessary to make approximately 7 samples, 2 inches in diameter and 2-1/2 inches high was calculated to be 1855 grams. The optimum moisture contents for the three asphalt contents were obtained from Pennell. They were as follows:

% MC-3	% Water
3	14.5
5	13.0
7	11.0

The weights of sand, water, and MC-3 necessary for 1855 grams of mix were calculated, and the results are as shown below:

% MC-3	Weight of Sand-gm.	Weight of Water-gm.	Weight of MC-3-gm.	Total Weight-gm.
3	1579	229	47	1855
5	1572	204	79	1855
7	1572	173	110	1855

This weight was sufficient for 6 compacted specimens and a moisture content sample.

2. The asphalt was heated to a temperature of 170 to 180 F in a closed drum.

3. The mixing pan was placed on the scale, the scale set at zero, and the calculated weight of sand was added.

4. The calculated weight of water was then added with a sprinkler and the sand was mixed with a spatula for about 3 minutes.

5. The pan, sand, and water were reweighed and the calculated weight of MC-3 was then added.

6. The mixture was then agitated with paddle-like strokes of the spatula for about 2 minutes and finally mixed by hand for about 3 minutes.

7. The prepared mixture was covered with a polyethylene sheet to minimize moisture losses during specimen preparation.

8. 265 grams of the mix was weighed out and placed loosely in the 6 inch by 2 inch mold. The mold was initially supported so that the plunger extended about 1-1/2 inches into the mold and the bottom of the mold was 1-1/2 inches above the plunger base.

9. One blow was applied to the specimen with the compaction hammer to set the specimen in the mold. The mold support was then removed and 17 more blows applied. The specimen and mold were then inverted on the plunger and 18 blows were applied to the other face of the specimen.

10. The mold and specimen were then removed from the compaction plunger and a round steel bar 2 inches shorter than the mold was pushed in one end of the mold, forcing the specimen out the other end. The specimen was then carefully trimmed flush with the end of the mold with a spatula.

11. The specimen was then weighed and assigned a number.

12. The mold and plunger were cleaned and steps 8 to 11 were repeated for the five remaining specimens.

13. A volatile content sample of about 100 grams was obtained after the third specimen in each group had been made. Volatile content was taken as the weight loss, divided by the dry weight, after drying for 24 hours at 100 C.

14. The procedure outlined in steps 3 to 13 inclusive was carefully adhered to for each of the 8 groups of 6 specimens of 3, 5, and 7 per cent MC-3.

15. All 2 inch by 2 inch MC-3 stabilized specimens were then cured for 5 days in an oven at 100 F.

2. 2 inch Sand-Portland Cement Specimens

The optimum portland cement content for the McGinn Pit sand was 8 per cent and the optimum water content was 16 per cent.*

A mix sufficient for 6 specimens was prepared, consisting of 1600 grams sand, 128 grams cement, and 276 grams water. Specimens were prepared in the same way as the asphalt stabilized specimens except only 10 blows per face were applied in compacting the specimens.The smaller compactive effort was employed in an attempt to get the same density as in the sand-asphalt specimens. No initial moisture content samples were taken. All specimens were initially cured for 7 days in a room at saturation humidity and at a temperature of 70 F.

3. Triaxial Compression Test Specimens

The procedure in preparing the 2.8 inch by 6.5 inch specimens was similar to that used for the 2 inch diameter by 2 inch specimens, with the following exceptions:

* Determined by the Highways Division of the Research Council of Alberta.

1. Due to the larger size specimens, the mix for each specimen was individually prepared.

2. A compaction hammer having a face diameter of 2.8 inches, a weight of 10.3 pounds, and a free drop of 18 inches was used.

3. The specimens were compacted in five layers, with compaction from both ends. Each layer was scarified before the next layer of mix was added.

4. In an attempt to achieve uniform density, the blows were staggered on the different layers, with the bottom layer receiving 8 blows, successive layers 9, 10, and 11, and the top layer receiving 12 blows.

5. The specimens were oven cured at 100 F for 7 days, since this was the curing time employed by Pennell in his more comprehensive triaxial testing program.